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GEOTECHNICAL DESIGN REPORT
IRVING BRIDGE REPLACEMENT
OVER PUSHAW STREAM
OLD TOWN, MAINE

MDOT SOILS REPORT 2005-16C

Penobscot County
MaineDOT PIN 11043.00
Fed. No. AC-BH-1104(300)X

Submitted to:

Maine Department of Transportation
Bridge Program
Augusta, Maine

Submitted by:

Golder Associates Inc.
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August 25, 2005

Our Ref.: 043-6811

Ms. Kate Maguire, P.E.
Maine Department of Transportation -- Bridge Program
16 State House Station
Augusta, Maine 04333-0016

**RE: GEOTECHNICAL DESIGN REPORT
IRVING BRIDGE REPLACEMENT OVER PUSHAW STREAM
OLD TOWN, MAINE
MAINE DOT PIN 11043.00**

Dear Kate:

Golder Associates Inc. (Golder) is pleased to submit this final report summarizing our geotechnical evaluation for the Irving Bridge Replacement project over Pushaw Stream in Old Town, Maine. The final report addresses the Maine Department of Transportation's (MaineDOT's) review comments for our draft report dated 6/2/04. The report was completed in accordance with our Project Contract with MaineDOT executed 12/29/03, and the provisions of our General Consultant Agreement # U088040396 with MaineDOT.

The report presents the findings from the field investigation completed at the site, discusses our geotechnical evaluations, and provides recommendations for foundation design and construction of the proposed replacement bridge. As discussed in the report, a preliminary design plan and profile for this project was not completed by MaineDOT at the time this report was prepared. When this information is available, we request the opportunity to review the plan and profile and confirm or modify as appropriate the recommendations provided in the report.

We appreciate the opportunity to assist MaineDOT with this interesting project. If you have any questions or require additional information, please contact Mark Peterson at 373-1520.

Very truly yours,

GOLDER ASSOCIATES INC.

Mark S. Peterson, P.E.
Senior Consultant

Peter C. Conti, P.E.
Principal

STATE OF MAINE
MARK S. PETERSON
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REGISTERED
PROFESSIONAL ENGINEER

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GEOTECHNICAL DESIGN SUMMARY

The Maine Department of Transportation (MaineDOT) intends to replace Irving Bridge over Pushaw Stream in Old Town, Maine with a one or two-span bridge founded on pile-supported integral abutments. The new bridge will be approximately 150 ft. long (70 ft. longer than the existing bridge). Preliminary design information is not currently available regarding proposed superstructure type, and approach roadway plan and profile. The purpose of this report is to provide a description of subsurface soil and groundwater conditions at the proposed bridge foundations and discuss geotechnical recommendations for proposed substructures.

Foundation Considerations: H-pile sections are recommended for support of integral abutment foundations. Piles at the north abutment are expected to be driven to bedrock refusal about 60 ft. below final grade, and axial capacity should be governed by the allowable structural capacity. Piles at the center pier and south abutment are not expected to penetrate a dense soil layer with cobbles and boulders present at about 50 to 60 ft. below grade and will need to derive capacity from end-bearing and friction resistance in soil. Geotechnical capacity will govern pile capacities at the south abutment and center pier. The subsurface investigation shows that the frequency of cobbles and boulders increases toward the southern bridge foundation locations. The piles should be made from ASTM A572, Grade 50 steel, and all piles should be fitted with prefabricated cast steel tips. An HP 14x89 section, or larger, is recommended for this project to account for high driving stresses. The allowable structural capacity for this pile section is 326 kips. The allowable geotechnical capacity will depend on the depth to which the pile can be driven. At the south abutment, an allowable geotechnical capacity for an HP 14x89 pile could vary from about 145 kips to 240 kips depending on pile depth achieved. At the center pier the allowable geotechnical capacity could vary from about 125 to 160 kips. Wave equation analyses are required to assess pile capacity and driveability. Dynamic pile testing is required to verify driven capacity.

Bridge Abutment Walls and Wingwalls: For cast-in–place integral abutments and wingwalls, passive earth pressures should be applied to the back face of the wall for wall design. Type 4 soil backfill, per the MaineDOT Bridge Design Guide, is recommended for the south abutment. A passive earth pressure coefficient, K_p, equal to 7.3 should be used for Type 4 soil with the following design properties: $\phi = 32$ degrees; $\delta = 2/3\phi$; and $\gamma = 125$ pcf. Type 5 soil should be considered for abutment wall backfill at the north abutment. Type 5 soil design parameters include: $K_p = 11.1$; $\phi = 36$ degrees; $\delta = 2/3\phi$; and $\gamma = 135$ pcf. At both abutments drainage features are required at the back face of the wall consisting of French Drains with weep holes, underdrain pipes wrapped with filter stone/geotextile, or geocomposite drainage materials.

Frost Depth: Foundations supported on subgrade soils should be founded a minimum of 6.2 ft. below finished exterior grade for frost protection.

Scour: To evaluate scour in the stream channel a D_{50} particle size of 0.62 mm should be used for soils from 0 to 4 ft. deep, and a $D_{50} = 0.58$ mm should be used for soils deeper than 4 ft.

Approach Design: Subgrade excavations at both abutments should be extended down to overexcavate the compressible soils and replace them with Granular Borrow for Underwater Backfill. The bottom of the overexcavated area at the south and north abutments is estimated to be about elev. 100 ft. If these materials are not planned to be removed, the stability and settlement behavior of widened shoulder fills should be evaluated when design grades are established. Within the existing roadway section at both proposed abutment areas subbase sand and gravel fill materials extend about 4 ft. bgs. These materials could be used for support of the new roadway sections. Drainage provisions should be included in the approach roadway design, particularly for the north abutment approach.

Settlement: Assuming potentially compressible soils in new abutment areas will be removed to about elev. 100 ft., the roadway approach settlements will be negligible at the abutment walls and downdrag loads on abutment piling can be ignored. Additional abutment settlement will be limited to the elastic compression of the pile foundations. In the event potentially compressible soils in new abutment areas are not planned to be removed, settlement magnitudes of approach fills and the attendant downdrag loads on abutment piling should be evaluated when design grades are established.

Seismic Design: The Irving Bridge site is located in a Seismic Performance Category (SPC) A area and a seismic analysis for foundations and substructures is not required.

Construction Considerations: The presence of cobbles and boulders in the outwash deposits could present difficulties in driving piles for foundations and sheet pile for cofferdams. The risk of these difficulties is expected to be higher at the south abutment than at the center pier or north abutment, based on the conditions encountered at the borings. Cobbles and boulders were initially encountered at the south abutment boring at a depth of about 14 ft. (elev. 98 ft.), about 46 ft. below mudline at the center pier boring (elev. 48 ft.), and were not encountered at all at the north abutment boring. For foundation piles, provisions should be included in the construction contract to allow the contractor to advance past obstructions if encountered in the upper soil strata (within about 25 ft. of ground surface, or a maximum depth of elev. 85 ft.) by using predrilling or spudding methods. The contractor should be required to obtain the Construction Resident's (resident's) approval for predrilling or spudding below elev. 85 ft.

The soils underlying the site are granular with high permeability. Excavations below the groundwater level should be designed to properly control seepage pressures to avoid developing bottom heave that could cause a loss of foundation soil strength.

1.0 INTRODUCTION

This report summarizes the results of Golder Associates Inc.'s (Golder's) subsurface investigation and geotechnical design for the replacement of Irving Bridge in Old Town, Maine. The existing bridge is located on Route 16 over Pushaw Stream approximately 0.4 miles north of the intersection of Routes 16 and 43 in Old Town, Maine, as shown on Figure 1. The purpose of the investigation was to explore subsurface soil and groundwater conditions at areas where new foundations are planned for the replacement bridge and develop geotechnical design criteria for new foundations and earth retaining structures. At the time this report was prepared a preliminary plan and profile of the replacement bridge had not been developed by the Maine Department of Transportation (MaineDOT), and our understanding of the proposed design was based on discussions with MaineDOT personnel. Our work was completed in accordance with our Project Contract with the MaineDOT executed 12/29/03, and the provisions of our General Consultant Agreement # U088040396 with MaineDOT.

2.0 PROJECT DESCRIPTION

Site topography and the arrangement of the existing Irving Bridge relative to Pushaw Stream is shown on Sheet 1 in Appendix D. The existing bridge, constructed in 1937, is an 81 foot (ft.) long, 22 ft. wide, single span, steel pony truss structure supported on concrete capped, stacked stone abutments. Prior to 1937 a previous bridge was supported directly on the stacked stone abutments. As part of the 1937 construction cast-in-place concrete caps were placed on each stone abutment to raise the bearing support grade for the bridge truss about 2 ft. According to plans for the 1937 construction provided by MaineDOT[1], the stone abutments are about 11.5 ft. in height; however, no information is available concerning foundation support conditions. Since 1937 the bridge has been painted repeatedly and a new concrete deck with an integral concrete wearing surface was built in 1991. The current condition of the substructure is considered "fair" by MaineDOT, including some moderate concrete scaling on the north abutment.

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[1] Two drawings provided by MaineDOT titled "Irving Bridge over Pushaw Stream, in the city of Old Town, Penobscot County", prepared by State Highway Commission, Bridge Division, Augusta, ME, Sheet 1 (Survey) and Sheet 2 (Substructure), June 1937, File Nos. 28-116 and 28-115.

We understand a complete replacement bridge is planned because the existing bridge is too narrow for current standards and the structure presents an undesirable constriction in Pushaw Stream. The replacement bridge is planned to be widened to 32 ft. and lengthened to approximately 150 ft. At this time MaineDOT is considering both a single span and a two-span replacement bridge. The new bridge abutments are expected to be located roughly 30 to 45 ft. behind the existing abutments. Thus, the existing stone abutments and portions of the approach roadways will be removed. Pile supported integral abutments are planned for the new bridge. If a two-span bridge is selected, the center pier could conceivably be supported on either a pile supported mass pier or a pile-bent pier. We understand the horizontal alignment of the proposed bridge will be approximately the same as the existing bridge, and vertical grades will be moderately raised (about 1 ft. at the south abutment and 2.5 ft. at the north abutment). The arrangement of fill slopes for widened shoulders in bridge approach areas were not available to Golder when this report was prepared.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were investigated by drilling three test borings at the locations shown on Sheet 1. Test borings BB-OTPS-101 and BB-OTPS-103 were drilled at the approximate locations of the proposed south and north abutments, respectively. Test boring BB-OTPS-102 was drilled from the deck of the existing bridge at the approximate location of a center pier for the replacement bridge, if needed. Borings BB-OTPS-101 and BB-OTPS-102 were performed by Northeast Diamond Drilling, Inc. of Brunswick, Maine from 1/5/04 to 1/20/04 using a truck-mounted CMT Model 75 drill rig. Boring BB-OTPS-103 was drilled by the MDOT drilling crew on 1/20/04 and 1/21/04 using a truck-mounted CME Model 45c rig. A Golder geotechnical engineer was present throughout the field program to log the conditions encountered and determine protocols for soil sampling and in-situ testing.

The borings were drilled to depths ranging from about 52 feet (ft.) to 80 ft. below ground surface (bgs) using wash boring methods and 4-inch driven casing. The boring drilled at the possible new center pier location (BB-OTPS-102) was drilled from the deck of the existing bridge which was about 10.7 ft. above the ice surface on Pushaw Stream during drilling and 18.5 ft. above the river bottom. At borings BB-OTPS-101 and 102 the presence of cobbles and boulders required telescoping down to 3-inch casing at about elevations 51 ft. and 44 ft., respectively, in efforts to advance the borehole. Soil samples were generally obtained at 5-ft. intervals using Standard

Penetration Test (SPT) procedures in accordance with ASTM D1586. Closer sample intervals were used in the upper 15 ft. of the abutment borings to more accurately assess the transition from fill materials to native soils. At boring BB-OTPS-103 a field vane shear test was conducted at 11.0 ft. bgs where a layer of clayey silt was encountered. The vane shear test was conducted with MaineDOT vane equipment in accordance with MDOT vane shear testing procedures[2]. Where stiff silty clay materials were encountered, the unconfined compressive strengths of the soils were approximated in the field using a pocket penetrometer on SPT samples. Rock coring was performed through boulders in boring BB-OTPS-101 and into bedrock at boring BB-OTPS-103 using an NQ double-tube core barrel. Rock Quality Designations (RQD) of the recovered bedrock samples were measured in the field and are reported on the boring logs. The boring locations and ground surface elevations were surveyed by an MaineDOT survey crew after the field program was completed.

Details of the drilling methods, field data obtained, and descriptions of the soil, rock and groundwater conditions encountered are presented on the boring logs presented in Appendix A. The logs are also shown on full size Sheet 2 at the end of this report.

4.0 LABORATORY TESTING

Geotechnical laboratory testing was limited to four moisture content determinations and three grain size analyses. Test results are presented in Appendix B.

5.0 SUBSURFACE CONDITIONS

Surficial soil conditions in the project area are mapped as sand and gravel outwash deposits and glacial till by the Maine Geological Survey[3]. The general soil stratigraphy observed at the borings consisted of the following, listed with increasing depth below the existing ground surface/stream bottom:

- *Fill* (absent at BB-OTPS-102) – Sand and gravel road fill overlying silty sands.
- *Alluvial and Glaciomarine deposits* – Mixtures of loose sand, silty sands and sandy silts with occasional organic materials (roots/branches) overlying a thin layer of clayey silt.

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[2] Maine Department of Transportation, "Vane Shear Testing Procedures", June 2001.
[3] Borns, Jr., H.W. and Thompson, W.B., "Reconnaissance Surficial Geology of the Orono Quadrangle, Maine", Open File No. 81-6, Maine Geological Survey, Department of Conservation, Augusta, ME, 1981.

- *Glacial Outwash* – Stratified layers of medium dense uniform sands and dense to very dense sandy gravels with cobbles and boulders. The presence of cobbles and boulders appears to increase in a southerly direction across the project area.
- *Glacial Outwash and Glacial Till Inclusions* - Dense to very dense sandy gravels and silty gravels, with cobbles and boulders.
- *Glacial Till* - A distinct layer of very dense glacial till was encountered only at boring BB-OTPS-101 at a depth of about 69 ft. bgs.
- *Bedrock* - The Kenduskaeg Unit, a phyllite and metasiltstone, was encountered only at BB-OTPS-103. The bedrock surface drops from north to south and was below the drilling depth for BB-OTPS-101 and 102.

A profile of interpreted subsurface stratigraphy is shown on Sheet 1. The following sections describe the encountered soil layers in more detail.

Fill: Fill materials were encountered at each abutment area directly beneath 5 to 6 inches of asphalt pavement. An upper fill layer extending to about 4 ft. bgs was interpreted to represent road base and subbase material, and consisted of coarse to fine sand with little gravel. At the south abutment (BB-OTPS-101) an underlying layer of sand with little gravel and silt extending to about 7 ft. bgs was interpreted to be a subbase fill material. This lower fill did not appear to be present at the north abutment. SPT N-values ranged between 70 and 83 in the upper layer indicating a very dense consistency, and between 20 and 44 in the lower layer at the south abutment indicating medium dense to dense conditions.

Alluvium: Alluvial deposits were encountered at all three boring locations. This layer was interpreted to be about 5 ft. thick at the abutments and about 4 ft. thick at the pier. Alluvium at the abutments included interbedded layers of loose to medium dense silty sand, sandy silt, sand, and silt with little gravel, clay and traces of organic materials (typically root hairs, decaying tree roots and/or limb fragments). In the abutment areas the SPT N-values in the alluvium varied from about 5 to 9 indicating loose conditions. An unconfined strength of 1.0 tons per square foot (tsf) was measured with a pocket penetrometer in a silty sample with little clay at about 11 ft. bgs at the south abutment. Moisture contents varied from about 23% to 29%.

At the stream bottom in the center pier area, the alluvial soils were interpreted to be about 4 ft. thick, and consisted of medium to find sand with some gravel and trace silt. An N-value of 16 was obtained from the one sample retrieved from this zone, indicating medium dense conditions. A grain size analysis of this material (see Appendix C gradation curve) indicates it is gap-graded with about 65% sand, 9% fines (silt and clay), a uniformity coefficient of 10.0, and a D_{50} of 0.65 mm.

Glaciomarine Deposits: A thin layer of marine clayey silt about 2 to 3 ft. thick was encountered at both abutment areas. At the south abutment this layer was encountered at a depth of about 12 ft. (elev. 99.8 ft.) and consisted of very stiff to hard olive gray clayey silt, which is commonly interpreted as the upper over-consolidated zone of the Presumpscot Formation present along the Maine coastal plain. Unconfined strengths were measured with a pocket penetrometer to be over 5 tsf at the surface of this stratum, dropping to about 2.5 tsf at the base. A moisture content of 23% was measured at this location.

At boring BB-OTPS-103 at the north abutment the clayey silt layer was encountered at about 10 ft. (elev. 102.9 ft.) and was very soft and gray. A vane shear test conducted at 11 ft. bgs indicated an undrained shear strength of 320 pounds per square foot (psf). This soft zone was interpreted to be only about 1.5 ft. in thickness. The clayey silt had a moisture content of 35% (sample 4D). Beneath the clayey silt layer at this boring a fragment of wood (assumed to be a tree limb) roughly 6 in. in diameter was encountered.

Outwash: The predominant materials underlying the site are glacial outwash sand and gravel deposits. As shown on Sheet 1 it appears that the stratigraphy within the outwash unit dips to the north and that two general material types are present. An upper zone of dark gray, uniformly graded sand with little gravel extends about 14 ft. (to elev. 84 ft.) below the base of the glaciomarine layer at the south abutment. This upper zone is thicker at the north abutment and extends about 37 ft. (to el. 63. ft.) below the glaciomarine layer. N-values in the upper zone ranged from 12 to 30, with an average of 20, indicating a medium dense consistency. A representative grain size analysis of this material indicated a particle distribution of 87% sand, 4% silt, a uniformity coefficient of 2.6, and a D_{50} of 0.58 mm. The lower outwash zone encountered is roughly 18 to 19 ft. thick, thinning to about 7 ft. at the north abutment, and consists of a sandy gravel/gravelly sand with little silt and variable cobble and boulder content. N-values in the lower zone ranged between 25 and 74 with an average of 51, indicating a dense to very dense consistency. A representative particle distribution of this material was found to contain about 26% gravel, 65% sand, 9% silt, a uniformity coefficient of 10.0, and a D_{50} of 0.65 mm. The bottom of the lower outwash materials was observed to vary from about elev. 64 ft. at the south abutment to about elev. 56. ft. at the north abutment.

Outwash with Glacial Till Inclusions: A layer of very dense sandy gravel with little to some silt and with cobbles and boulders was encountered below the outwash layer that is interpreted to be a glacial fluvial transition zone. As seen on Sheet 1, the surface of this layer appears to slope down from south to north similar to the overlying stratigraphy. At the south abutment this layer was encountered at about elev. 64 ft. and was interpreted to be about 21 ft. thick. The surface of this layer appeared to drop to about elev. 59 ft. at the center pier area, and was not penetrated at the bottom of the boring at about elev. 43 ft. for BB-OTPS-102. This layer was not encountered at the north abutment due to the higher bedrock surface in that area. N-values across the stratum ranged from 48 blows per foot to 123 blows per 1 inch, but were likely influenced by coarse gravels, cobbles and boulders.

A significant characteristic of this layer is the apparent increased frequency of cobbles and boulders throughout the deposit, and particularly at the south abutment where difficult drilling conditions were encountered at boring BB-OTPS-101. An increase in the cobble and boulder frequency was encountered below about elev. 56 ft. at the south abutment and about elev. 48 ft. at the center pier. The boring logs show a detailed delineation of cobble and boulder conditions at the boring locations.

Glacial Till: A layer of glacial till was encountered near the bottom of the south abutment boring at a depth of about 69 ft. bgs. The till consisted of very dense gray silt, with some sand, little gravel, little clay, and contained cobbles and boulders. The layer thickness was not determined and extended beyond the depth of the boring. This layer was not encountered at the center pier area (it could be present beneath the bottom of the boring) and was not present at the north abutment area.

Bedrock: The bedrock surface was encountered only at the north abutment boring BB-OTPS-103, and appears to be dipping down to the south as shown on Sheet 1. At the north abutment the bedrock surface was encountered at about elev. 56 ft. The rock was identified from rock core samples as the Kenduskeag Unit, consisting of medium to dark gray phyllite and metasiltstone, with bands of quartzite and calcite. The RQD of the rock measured from two core runs were 29.1% and 32.5%, indicating poor quality.

Groundwater Levels: Water levels measured in the open boreholes during drilling in January 2004 were at about elev. 103 ft. (8.6 ft. bgs at the south abutment and 9.6 ft. bgs at the north abutment), which was about 1 ft. above the ice level on Pushaw Stream during the same period. Since both of these borings were located roughly 65 ft. from the river, a water table gradient of roughly 0.015 was present during the drilling program. Due to the relatively high permeability of the soils at the abutment areas, it is expected that groundwater levels will fluctuate closely with variations in the river level.

6.0 FOUNDATION ALTERNATIVES

Pile foundations are feasible for the abutments and a center pier given that the support soils are granular and of sufficient depth to satisfy requirements for depth of fixity as stated in the MDOT Bridge Design Guide[4] (BDG). Spread footing support for the abutments is also feasible provided the footings bear directly on native outwash deposits or on structural fills placed over the outwash deposits.

We understand the designer prefers to use an integral abutment or semi-integral abutment design for the replacement bridge. Pile supported integral abutments are feasible, however, BDG criteria prohibit the use of spread footing foundations to support integral abutments for this bridge. Both pile foundations and spread footings are considered feasible for conventional abutments supporting a simply supported structure for this bridge.

Integral Abutments Founded on H-Pile Foundations: Driven H-piles are a viable foundation type for integral abutments at this site. At the north abutment the pile can be designed as an end bearing pile driven to bedrock refusal. At the south abutment driven piles are not expected to penetrate a very dense soil layer with cobbles and boulders encountered at roughly elev. 56 ft. (about 56 ft. bgs). Accordingly, these piles would need to be designed for end-bearing and friction resistance in soil. Relatively thick H-pile sections with reinforced tips are needed to withstand driving stresses expected when encountering randomly located cobbles and boulders throughout the outwash strata. Steel pipe piles would be expected to encounter considerable difficulty penetrating cobbles and

[4] Maine Department of Transportation, "Bridge Design Guide", prepared by Guertin Elkerton & Associates, August 2003.

boulders in the outwash deposits at shallower depths at the south abutment area, and are not considered a feasible pile type at that location.

Center Pier Founded on Piles: Both a mass pier and a pile bent pier appear to be possible at this site depending on the bridge designer's assessment of ice conditions on Pushaw Stream. If severe ice conditions are expected, a pile bent pier option would be eliminated. Driven H-piles and pipe piles are feasible options for supporting the center pier. Pipe piles are considered feasible because cobbles and boulders were not encountered in the upper outwash deposits at boring BB-OTPS-102. Both pile types would need to be designed as soil supported end bearing and friction piles with limited, if any, penetration below the cobble and boulder zone encountered at roughly elev. 48 ft. (about 46 ft. below the stream bottom). Considering the risk of encountering shallow boulders and a preference to avoid using two types of piles for this project, only an H-pile foundation for the center pier is addressed in this report.

Conventional Abutments Supported on Spread Footings: Spread footings could be used to support a conventional abutment wall for a simply supported abutment, however, the footing would need to be founded at least 2 ft. below the scour depth in accordance with BDG criteria. Since the scour depth is assumed to be greater than 20 ft. below the existing ground surface at the abutments and greater than 10 ft. below the groundwater/stream level, it is assumed costs associated with deep excavation, cofferdams and dewatering would be excessive for this option. Considering these cost concerns and the designers' preference for integral abutments at this site, spread footing foundations are not addressed in this report.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

7.1 Driven H-Pile Foundations

On the basis of discussions with MaineDOT we assume the replacement bridge will be constructed with integral stub abutments supported on driven H-piles. If a two span bridge is planned, we assume the center pier will also be supported on driven H-piles. The presence of cobbles and boulders in the outwash deposits underlying the site, particularly at the south abutment area, will probably result in hard driving conditions during pile installations. Accordingly, we recommend the piles be comprised of rolled-steel sections of ASTM A572, Grade 50 steel, with a minimum yield stress of 50 kips per square inch (ksi) and that all piles be fitted with prefabricated cast steel tips conforming to MaineDOT Standard Specification Section 501.10 to reinforce and protect the base of the piles. An HP 14x89 section, or larger, is recommended for this project to account for high driving stresses. The allowable structural capacity for this pile section is 326 kips using a factor of safety (FS) of 4.0 in accordance with BDG criteria for integral abutment piles.

The allowable geotechnical capacity will vary with depth for each abutment location and the center pier due to different soil conditions. At the north abutment we expect the piles can be driven to refusal in the bedrock, the surface of which was encountered at about elev. 56 ft. For this condition the pile capacity will be governed by its structural capacity. At the south abutment and the center pier the piles are expected to encounter refusal resistance in the layer of interbedded outwash and glacial till with cobbles and boulders, and the piles will derive capacity from soil end bearing and side friction resistance. **Accordingly, geotechnical capacity will govern the axial pile capacity at these locations and will increase with increasing pile penetration.**

Figure 2 presents a plot of estimated pile capacity versus tip elevation for an HP 14x89 pile driven at each of the proposed abutments and at the center pier location. The allowable static capacities were estimated using the Nordlund Method to calculate an ultimate resistance, and applying a FS = 2.25 assuming wave equation analyses will be completed and the piles will be dynamically tested (per BDG criteria). To assess the applicability of the Nordlund Method in similar pile-soil conditions Golder reviewed static capacity calculations and dynamic testing data for MaineDOT's bridge replacement at New Bridge at Canaan, Maine (PIN 10103, piles driven 8/27/03). The findings from this review showed good agreement between static capacity calculations using the Nordlund Method and the results of wave equation analyses and dynamic testing data.

As presented in the pile capacity calculations in Appendix C, pile friction was neglected within about 15 ft. of finished grade at the abutments and within about 7 ft. of the existing stream bottom at the center pier to account for pile cap dimensions and construction considerations (soil excavation/replacement, pile driving vibrations, etc.). The calculated pile friction resistance at the center pier was determined assuming 4 ft. of scour occurs at this location. Calculated scour depths will be made by MaineDOT subsequent to the preparation of this report. If the calculated scour depth exceeds 4 ft. at the center pier, pile capacities should be re-evaluated. In accordance with recommendations discussed in Sections 7.6 and 7.7, down drag loads were not applied to the piles assuming upper loose/soft alluvial soils and organic materials at the abutment areas will be excavated prior to pile installations.

The pile lengths and axial capacities that can be achieved during construction will depend on the cobble and boulder obstructions encountered during installation. The minimum embedment length to achieve fixity for an end-bearing HP 14x89 pile is 15 ft. as stated in the BDG (Table 5.5). For an HP 14x89 driven into medium dense outwash deposits, Table 5-8 in the BDG indicates a depth of fixity of 27 ft. would be needed to resist the maximum lateral load capacity. For either case satisfactory soil thickness exists to provide lateral pile support. Table 1 below summarizes our qualitative estimate of the likelihood of varying pile tip penetrations being achieved during pile driving based on the cobble and boulder conditions encountered at the boring locations. More frequent cobbles and boulders were encountered at the south abutment and predrilling or spudding may be required to penetrate upper obstructions. Predrilling or spudding should not be allowed below about elev. 85 ft. without approval of the Construction Resident (resident). Vibratory hammers should not be allowed for any pile installations. If impenetrable cobble or boulder obstructions are encountered at greater depth, the pile capacity should be reduced in accordance with Figure 2 and/or a supplemental pile(s) should be driven at a horizontal distance of at least three (3) pile diameters away from the original pile unless more stringent criteria is established by the designer.

Wave equation analyses and dynamic pile testing should be performed for this project and will be a critical requirement in assessing appropriate pile driving equipment, developing a driving criteria, and confirming driven capacity. Wave equation analyses should be performed prior to construction to assure that an undersized hammer is not selected for pile installations and that the pile section can withstand the expected driving stresses. Accordingly, it is recommended that the wave equation analyses be performed to assess driveabiltiy and to produce a bearing graph. Protocols for pile testing discussed in Section 5.7.5 of the BDG should be followed. **Due to differing subsurface conditions at the proposed foundation areas, we recommend that separate wave equation analyses be performed for each foundation area.** At least one pile at each abutment and one pile at the center pier should be subject to dynamic testing during construction with a Pile Driving Analyzer. The dynamic testing should be performed from the start to the end of driving for each pile tested, and restrike dynamic tests should be conducted 24 hours after initial driving at all test pile locations. Driving stresses in the piles should be limited to 90% of the yield strength (45 ksi) in compression and tension. We recommend that a CAPWAP post-driving analysis be conducted for each dynamic test.

TABLE 1:
Variations in Estimated Maximum Allowable Geotechnical Pile Capacity With Driven Tip Elevation – HP 14x89
Irving Bridge Replacement – Old Town, Maine

Location	Likelihood of Achieving Driving Depth(1)	Driving Depth(2) (ft)	Pile Tip Elevation (ft-msl)	Allowable Capacity(3) (kips)
South Abutment	Expected(4)	49	64	145
	Probable	57	56	210
	Possible	61	52	240
	Not Likely	>61	<52	>240
Center Pier(5)	Probable	42	48	125
	Possible	46	44	160
	Not Likely	>46	<44	>160
North Abutment	Little Difficulty	52.5	63	110
	Probable	59.5	56	170
	Likely Driven into Bedrock	60.5	55	326(6)

Notes:
1. Based on conditions at test borings.
2. Depth below assumed finished grade (el. 113.0 ft. at south abutment, el. 115.5 ft. at north abutment, el. 90.4 ft. at center pier assuming 4.0 ft. of scour occurs).
3. Allowable geotechnical capacity determined with Nordlund Method using a FS = 2.25.
4. Cobble and boulder obstructions may be encountered within upper soil strata at south abutment, but are expected to be penetrated during pile driving.
5. Allowable capacities at center pier calculated based on assumed scour depth of 4.0 ft. If scour depth determined by MaineDOT is not 4.0 ft., the allowable capacities should be re-evaluated.
6. Governed by structural capacity for pile driven to bedrock refusal.

7.2 Center Pier Foundation Support Considerations

As discussed in Section 6.0 both a mass pier and a pile bent pier are considered feasible, and steel H-piles or pipe piles could be used. Allowable axial capacities for HP 14x89 piles at the center pier location are discussed in Section 7.1. For scour protection of mass piers a deep seal should be placed at least 2 ft. below the design scour depth, or the piles should be designed for an unsupported length equal to the distance between the bottom of the seal and the design scour depth (BDG, pg 5-40).

If a pile bent pier is selected and H-Piles are used, the H-Piles would need to be encased for corrosion protection with a concrete-filled pipe pile from the pier cap to at least 10 ft. below the streambed or 2 ft. below the total scour depth (BDG, pg 5-45). If the encased section is also used for lateral load resistance, it should extend to the point of fixity. Pipe pile encasements used for corrosion protection should be coated with fusion-bonded epoxy paint extending to 2 ft. below scour depth. If the pipe pile encasements are used for lateral load resistance, the epoxy coating should extend to the depth of fixity.

7.3 Bridge Abutment Walls and Wingwalls

For integral abutments the materials used for wall backfill should, at a minimum, meet the gradation requirements for Granular Borrow Underwater Backfill (MaineDOT 703.19 Standard Specifications). More stringent gradation criteria should be considered for abutment wall backfill at the north abutment. We understand the Route 16 road section immediately north of Irving Bridge was recently rebuilt in part to accommodate subgrade groundwater seepage conditions. Considering the guidance provided in Section 5.4.2.11 of the BDG, we suggest that wall backfill conforming to Gravel Borrow specifications (MaineDOT 703.20) be considered at the north abutment. At both abutments a positive drainage feature is needed at the back face of the wall to prevent the build-up of hydrostatic pressure behind the wall. Drainage can consist of French Drains with weep holes, underdrain pipes wrapped with filter stone/geotextile, or geocomposite drainage materials.

For cast-in–place integral abutments and wingwalls, passive earth pressures should be applied to the back face of the wall for wall design in accordance with Section 5.4.2.9 of the BDG. A passive earth pressure coefficient, K_p, equal to 7.3 is recommended with Granular Borrow Underwater Backfill

(Type 4 soils, BDG Table 3-3, pg 3-3), and backfill design properties should include $\phi = 32$ degrees, $\delta = 2/3\phi$, and $\gamma = 125$ pcf. For Gravel Borrow wall backfill (Type 5 soils) recommended design parameters include: $K_p = 11.1$; $\phi = 36$ degrees; $\delta = 2/3\phi$; and $\gamma = 135$ pcf. If an approach slab is not used, additional earth pressures from traffic loads should be treated as a surcharge load equal to the pressure applied by an equivalent height of soil, H_{eq}, as defined in Section 3.6.8 of the BDG (pg 3-9).

7.4 Frost Depth

According to Section 5.2.1 of the BDG, the Irving Bridge site in Old Town has a design freezing index of 1800 F degree days. Given that the shallow soils present at the site are predominantly coarse grained and have a moisture content on the order of 23%, the design frost depth is 74.5 inches (6.2 feet) according to Table 5-1 in the BDG. Foundations supported on subgrade soils should be founded a minimum of 6.2 ft. below finished exterior grade for frost protection.

7.5 Scour

The design scour depth for the site will generally depend on the hydraulic characteristics of Pushaw Stream, the configuration of the channel at the replacement bridge, flow vortices at piers and abutments, and the streambed soils. Most of these factors will be evaluated as part of the scour analysis completed by the designer. Streambed soils in the center pier vicinity were examined at Boring BB-OTPS-102, where grain size analyses were completed on a sample from the 0-2 ft. depth interval (S-1, interpreted to be an alluvial deposit)), and on a sample from the 5-7 ft. interval (S-2, interpreted to be an outwash deposit)). The gradation distribution curve for Sample S-1 is shown in Appendix B and is described as a gap graded medium to fine sand, with some gravel and trace silt. In accordance with criteria discussed in Hydraulic Engineering Circular No. 18[5], the D_{50} particle size is commonly used to assess the scour susceptibility of soils. The D_{50} particle size for this sample was 0.65 millimeters (mm). The transition to the underlying more uniformly graded outwash deposit is estimated to be about 4 ft. below the streambed surface. The outwash at this location is described

[5]Ayres Associates, "Evaluating Scour At Bridges - Third Edition", Hydraulic Engineering Circular (HEC) No. 18, sponsored by Federal Highway Administration, Office of Technology Applications, HTA-22, Publication No. FHWA-IP-90-017, November 1995.

as a coarse to medium sand, with trace gravel and trace silt. The D_{50} size for this sample was 0.58 mm.

Scour countermeasures can include a riprap blanket at the abutment slopes. We understand riprap will not be used as a countermeasure at the center pier. Guidance for countermeasures is provided in Sections 2.3.11.2 and 2.3.11.3 of the BDG and is based on design flow velocities, slope angles, and channel characteristics. For any riprap application filter criteria must be satisfied against the underlying base soil in the streambed and bank to prevent erosion of the fine grained soil matrix. Soil filter layers or geotextiles can be considered for this purpose.

7.6 Approach Design

Information concerning the design of the approach roadway and embankments for the replacement bridge were not available to Golder when this report was prepared. Accordingly, an assessment of new shoulder fill stability or settlement was not performed. When additional design information is available we suggest shoulder stability and settlement issues be considered. Settlement from consolidation of thin layers of loose/soft alluvium and glaciomarine deposits is possible under the weight of additional road fill (see Section 7.7). However, since these materials are expected to be in close proximity to the base of proposed integral abutment structures, it is recommended that the subgrade excavation at both abutments be extended down to overexcavate the compressible soils and replace them with Granular Borrow for Underwater Backfill (703.19) compacted in accordance with Section 203.12 of MaineDOT's Standard Specifications. On the basis of conditions encountered at the borings, it is estimated the bottom of the overexcavated area would be at about elev. 100 ft. at both abutments. Since this depth of excavation is anticipated to extend below the groundwater level (measured at approximately elev. 103.4 ft. in January 2004), a cofferdam would be required.

In the event the subgrade excavations at the abutments are not planned to extend to about elev. 100 ft., or sufficient depth to remove the compressible soils encountered at the explorations, we recommend additional evaluations be conducted to assess the stability and settlement behavior of the widened roadway shoulder fill, and possible down-drag loads on pile foundations.

Within the existing roadway section at both proposed abutment areas it appears that subbase sand and gravel fill materials extend about 4 ft. bgs. These materials are not considered to be frost susceptible and could be used for support of the new roadway sections. As noted in Section 7.3, we understand the Route 16 road section immediately north of Irving Bridge was recently rebuilt in part to accommodate subgrade groundwater seepage conditions. Accordingly, drainage provisions should be considered in the approach roadway design, particularly for the north abutment approach.

7.7 Settlement

Settlement from consolidation of thin layers of loose/soft alluvium and glaciomarine deposits is possible under the weight of additional road fill. Potentially compressible soils were encountered from about 7 to 12 ft. bgs at the south abutment and from about 5 to 13 ft. bgs at the north abutment. Since the amount of additional fill planned is expected to be minor (1 to 2 ft.), corresponding settlements would be expected to be relatively low. However, since these materials are located within the zone of abutment wall and backfill excavations, it is recommended that the potentially compressible soils be removed to about elev. 100 ft. as discussed in Section 7.6. If these materials are excavated and replaced with Common Borrow, it is concluded that roadway approach settlements will be negligible at the abutment walls and downdrag loads on abutment piling can be ignored. Additional abutment settlement will be limited to the elastic compression of the pile foundations.

If the potentially compressible materials discussed above are not planned to be removed, we recommend an evaluation of shoulder fill settlement be conducted when the proposed approach fill grades are known.

1.1 Seismic Design

The Irving Bridge site is located in a Seismic Performance Category (SPC) A area because the horizontal acceleration coefficient is less than 0.09g per Figure 3-4 in the BDG. According to AASHTO Standard Specifications a detailed seismic analysis is not required for SPC A bridges. The MaineDOT Bridge Program does have a provision to require seismic analyses for *major and functionally important* SPC A bridges with two or more spans. The Irving Bridge is not considered a major bridge because it is not located on a National Highway System road. The bridge designer is responsible for determining if the bridge is considered functionally important. Based on our discussions with MaineDOT we understand seismic analyses will not be required for foundation design of this bridge.

7.9 Construction Considerations

Pile foundations should be constructed in accordance with Section 501 of the Standard Specifications. The presence of cobbles and boulders in the outwash deposits could present difficulties for pile driving installations for foundations and sheet pile installations for cofferdams. The risk of these difficulties is expected to be higher at the south abutment than at the center pier of north abutment based on the conditions encountered at the borings. Cobbles and boulders were initially encountered at about elev. 98 ft. at the south abutment boring, about elev.48 ft. at the center pier boring, and were not encountered at all at the north abutment boring.

For foundation piles, we recommend that provisions be included in the construction contract to allow the contractor to advance past obstructions if encountered in the upper soil strata (within about 25 ft. of ground surface, or a maximum depth of elev. 85 ft.) by using predrilling or spudding methods. Due to concerns regarding reduced frictional resistance for foundation piles if obstructions are cleared at lower elevations, we recommend the contractor be required to obtain the resident's approval for predrilling or spudding below elev. 85 ft. The following notes should be included on the plans:

The Contractor may encounter obstructions in the form of cobbles or boulders during pile driving operations. This condition is anticipated to be more prevalent at the south abutment area. The Contractor may clear obstructions from the ground surface to a pile tip elevation no deeper than elev. 85 ft. using predrilling or spudding methods. Clearing obstructions deeper than elev. 85 ft. likely will not be allowed. Contractor shall obtain Construction Resident's (Resident's) approval prior to clearing obstructions at deeper pile tip elevations. Criteria for predrilling and spudding are as follows:

A. *Predrilling: The Contractor may predrill the pile locations with a solid stem auger. The predrilled hole diameter shall not exceed 16 inches* (for an HP 14x89 pile). *Disposal of spoils from predrilling operations must meet Maine DEP approval. Predrilling shall extend* *to a depth no deeper than elev. 85 ft. unless approved by the Resident. Predrilling shall be in accordance with Section 501.04 of the Standard Specifications.*

B. *Spudding: The Contractor may clear obstructions by spudding. The spud shall consist of an H-pile section the same size, or smaller, than the production piles. Other spuds may be accepted, as approved by the Resident.. Spuds shall be driven in the production pile locations to a tip elevation not below 85 ft., unless otherwise approved by the Resident. Once driven, the spud shall be removed. In the event the Contractor cannot remove a spud, the Contractor shall bear all expenses for additional piles and any associated design changes.*

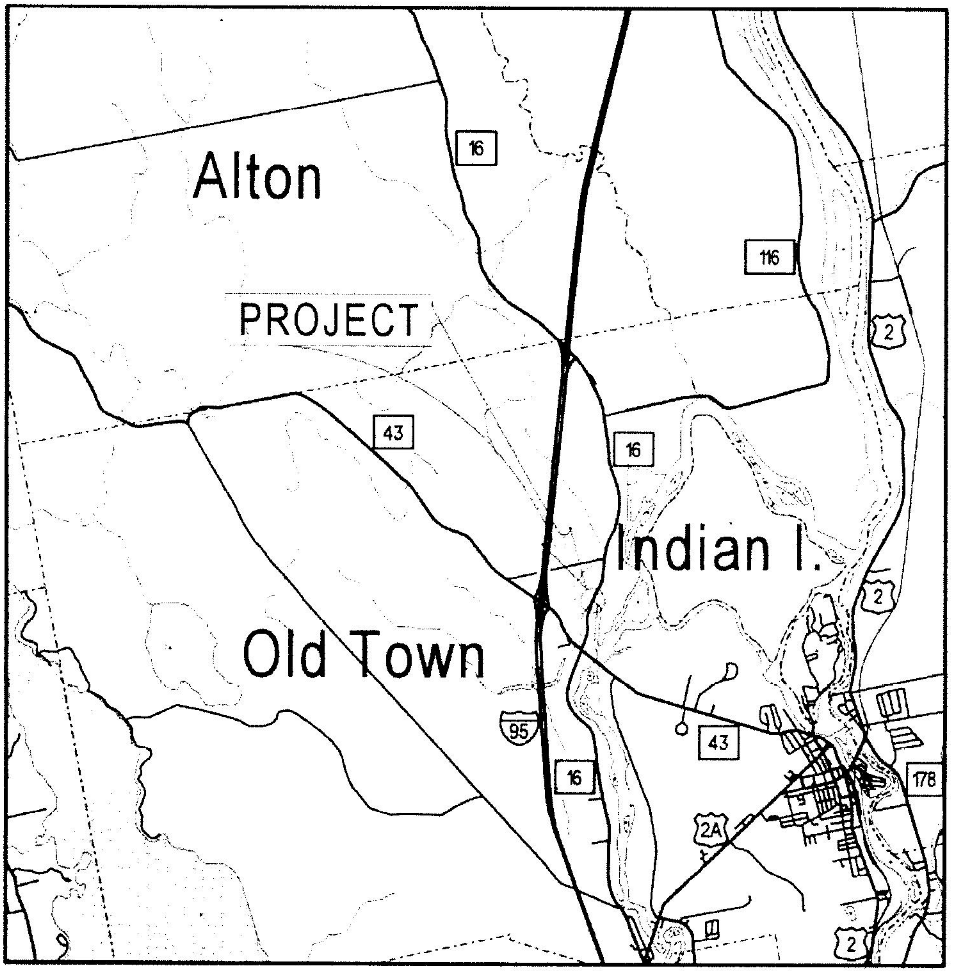
The cost of clearing obstructions shall be considered incidental to contract related pay items.

Cofferdams will be required for the center pier (if selected and a mass pier option is used), and may be needed for portions of the new abutment construction. For the center pier cofferdam the bottom seal should be designed in accordance with criteria provided in Sections 5.2.2 and 5.2.3 of the BDG. If abutment cofferdams are constructed and sheet piles are used, the sheet piles should be driven a sufficient depth below the bottom of the excavation, and/or appropriate dewatering methods should be employed to prevent heaving or strength loss of the foundation soils due to seepage pressures. Similar to the above discussion for foundation piling, it is expected that sheet pile installations may encounter cobble and boulder obstructions, particularly at the south abutment area, starting at a depth of about 14 ft. (elev. 98 ft.) below the existing ground surface. An impact hammer is expected to be needed in this area to advance the sheet piles to required depths. At the center pier area, cobbles and boulders may not be encountered during sheet pile driving based on the boring data. At the north abutment pieces of wood (tree limbs) were encountered about 12 ft. below the existing ground surface (elev. 101 ft.) which could also affect the installation of sheet piles. The design of bracing and dewatering systems for these temporary structures should be developed considering these site conditions, and should be designed by a professional engineer registered in the State of Maine.

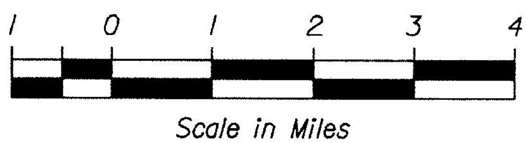
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the Irving Bridge in Old Town, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other warranty, express or implied, is made. Certain design information was not available to Golder at the time this report was prepared, and recommendations are provided in the report for additional evaluation when this information becomes available. In the event that any changes in the nature, design, loading conditions or location of the proposed project are planned from those described or referenced herein, Golder should be notified to review the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analysis and recommendations are based on findings from the field investigation, combined with an interpretation of soil and groundwater conditions encountered at discrete site locations. If variations from the conditions encountered during the investigation appear evident during construction, Golder should be notified so that we may review and verify or modify our recommendations as appropriate. We also recommend that we be provided the opportunity for a review of final design drawings and specifications in order that the earthwork and foundation recommendations are properly interpreted and implemented in the design and specifications.

FIGURES



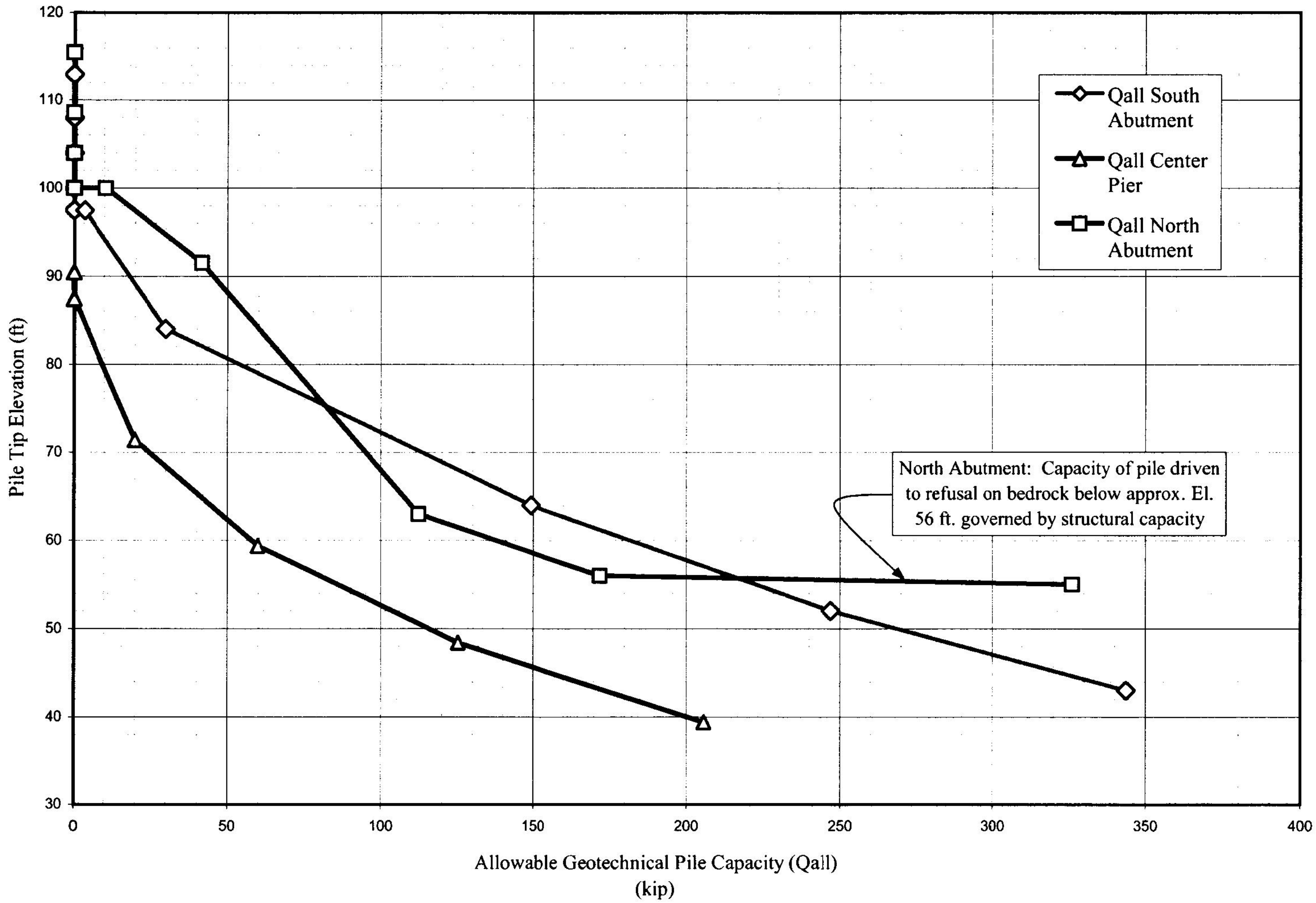
LOCATION MAP



Old Town
Irving Bridge
11043.00

Golder Associates	Project: IRVING BRIDGE REPLACEMENT Old Town, Maine	Title: Site Location Map		By: MSP Checked: PCC Date: 8/25/2005
	Client: Maine DOT	Proj. No: 043-6811	Scale: nts	Fig. No: 1

Figure 2: Allowable Geotechnical Capacity for HP 14x89 Pile at Irving Bridge by Nordlund Method



Project No: 043-6811	Title: ALLOWABLE GEOTECHNICAL CAPACITY HP 14X89 PILE	By:	MSP
	Irving Bridge Replacement, Old Town, Maine	Checked:	PCC
Figure No: 2	*Prepared for: Maine Department of Transporation*	Date:	8/19/2005
		Scale:	as shown

Golder Associates

APPENDIX A

BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project:	Irving Bridge Replacement crossing Pushaw Stream	**Boring No.:**	BB-OTPS-101
Location:	Route 16 Old Town, Maine	**PIN:**	11043.00

Driller:	Northeast Diamond Drilling	**Elevation (ft.)**	111.8 ft	**Auger ID/OD:**	
Operator:	R. Leonard	**Datum:**	NGVD	**Sampler:**	SPT split spoon. 2 ft. long
Logged By:	R. Bennett	**Rig Type:**	CMT Model 75	**Hammer Wt./Fall:**	140 lb / 30 in
Date Start/Finish:	1/5/04 - 1/8/04	**Drilling Method:**	Washed, Driven Casing	**Core Barrel:**	NQ, 2.0 in ID, dble tube
Boring Location:	Proposed South Abutment	**Casing ID/OD:**	4 in 0-61 ft, 3 in 61+	**Water Level*:**	8.64 ft. bgs 1/6/04

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	18/10	0.5 - 2.0	34/49/65	83	Cased	111.3		6 in. Asphalt pavement — 0.5	
									Gray, very moist, very dense, coarse to fine SAND, some gravel, trace silt, with petroleum odor. (Road Subbase Fill) — 2.0	
	2D	24/12	2.0 - 4.0	42/18/52/75	70		109.8		Dark gray, very moist, very dense, gravelly coarse to medium SAND, trace silt, with petroleum odor. (Road Subbase Fill) — 4.0	
5	3D	24/12	4.0 - 6.0	35/23/21/41	44		107.8		Olive brown, very moist, dense, medium to fine SAND, little gravel, little silt. (Suspected Subgrade Fill) — 7.0	
	4D	24/10	6.0 - 8.0	24/12/8/7	20					
							104.8		Olive brown, wet, medium dense, silty fine to medium SAND, trace fine gravel with occasional mottling. (Alluvium?) — 8.0	
	5D	24/6	8.0 - 10.0	7/3/25/6	28		103.8		Brown, wet, loose, coarse to fine SAND, little to some silt, little fine gravel to 1 in. size, trace organics. (Alluvium) — 9.0	
							102.8		Brown, wet, silty medium to fine SAND, little fine gravel, trace black organics (suspected decayed wood). (Alluvium) — 10.0	
10	6D	24/18	10.0 - 12.0	2/2/3/10	5		101.8		Olive gray, wet, medium stiff, SILT, some fine sand, trace to little clay, trace organics. Mixed with silty SAND, trace fine gravel. (Alluvium) PPT = 1.0 TSF — 12.0	WC = 29.0%
	7D	21/21	12.0 - 13.8	24/18/14/>62	32		99.8		Olive gray, wet, hard to very stiff, SILT, with little clay, becoming CLAYEY SILT, little fine sand, trace fine gravel. (Glaciomarine) 12-13 ft, PPT>5.0 tsf 13-14 ft, PPT=2.5-3.0 tsf — 14.0	WC = 23.0%
15	8D	4.5/4.5	14.0 - 14.4	>100 for 4.5"	na		97.8		Olive brown/gray, wet, SILT, some clay, little fine sand, little angular black fine gravel. (Glaciomarine) — 14.2	
							97.6		Probable cobble 14.2-14.5 ft. based on drilling action. — 14.5	
							97.3			
							95.9		15.9 — Probable cobble 15.9-16.2 ft. based on drilling action. — 16.2	
							95.6			
							92.8		19.0 — Probable cobble 19.0-19.8 ft. based on drilling action. — 19.8	
20	9D	24/10	20.0 - 22.0	43/10/8/11	18		92.0		Dark gray, wet, medium dense, coarse to fine SAND, little gravel. (Outwash)	
						34				
25	10D	24/0	24.0 - 26.0	10/10/10/7	20	28			No recovery	
						38				
						40				
						50				
						64	83.8		— 28.0	
							82.8		— 29.0	
30	11D	24/6	29.0 - 31.0	25/73/51/18	69	104			Gray, wet. very dense, gravelly coarse to fine SAND, trace silt. (Outwash)	

Remarks:

"*" Indicates pre-washed 4" casing blow counts
"**" Indicates pre-washed 3" casing blow counts
PPT = Pocket Penetrometer

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project:	Irving Bridge Replacement crossing Pushaw Stream	**Boring No.:**	BB-OTPS-101		
Location:	Route 16, Old Town, Maine	**PIN:**	11043.00		
Driller:	Northeast Diamond Drilling	**Elevation (ft.)**	111.8 ft	**Auger ID/OD:**	
Operator:	R. Leonard	**Datum:**	NGVD	**Sampler:**	SPT split spoon, 2 ft. long
Logged By:	R. Bennett	**Rig Type:**	CMT Model 75	**Hammer Wt./Fall:**	140 lb / 30 in
Date Start/Finish:	1/5/04 - 1/8/04	**Drilling Method:**	Washed, Driven Casing	**Core Barrel:**	NQ, 2.0 in ID, dble tube
Boring Location:	Proposed South Abutment	**Casing ID/OD:**	4 in 0-61 ft, 3 in 61+	**Water Level*:**	8.64 ft. bgs 1/6/04

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30						119			
						88			
						101			
						161			
	12D	24/7	34.0 - 36.0	42/16/19/35	35	169		Dark gray, wet, dense, angular to rounded GRAVEL, little medium to fine sand. (Outwash)	
35						196			
						144			
						140			
						175			
	13D	24/3	39.0 - 41.0	62/37/24/30	61	203		Dark gray, wet, very dense, GRAVEL, little coarse to medium sand, trace silt. (Outwash)	
40						360	71.8	Rock fragments in sampler tip. (Outwash) 40.0	
						279	70.8	Probable boulder 40.0 - 41.0 ft. based on drilling action. 41.0	
						170	69.3	42.5	
						159			
	14D	24/5	44.0 - 46.0	16/22/25/18	47	92*		Dark gray, wet, dense, coarse to medium SAND, little gravel. (Outwash)	
45						56*			
						98*			
						100*	64.3	47.5	
						150*	63.8 63.3	Possibly Glacial Till based on return of wash water pressure. 48.0	
						110*		Probable cobble 48.0-48.5 ft. based on drilling action 48.5	
50	15D	24/2	50.0 - 52.0	27/25/52/38	77	171		Dark gray, wet, subangular to rounded GRAVEL, some coarse to medium sand, trace silt. (Silty outwash with Glacial Till inclusions)	
						195			
						254			
						222			
	16D	24/3	54.0 - 56.0	42/47/14/30	61	198		Dark gray, wet, fine GRAVEL, some coarse to medium sand, trace silt and clay . (Silty outwash with Glacial Till inclusions)	
55						441			
						510	55.8	56.0 Increased cobbles and boulders mixed with gravel (Outwash)	
						261			
						205			
60						NA	52.8 52.3	59.0 Probable cobble 59.0-59.5 ft. based on drilling action.	

Remarks:

"*" Indicates pre-washed 4" casing blow counts
"**" Indicates pre-washed 3" casing blow counts
PPT = Pocket Penetrometer

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

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US UNITS

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		PIN:	11043.00

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Logged By:	R. Bennett	**Rig Type:**	CMT Model 75	**Hammer Wt./Fall:**	140 lb / 30 in
Date Start/Finish:	1/5/04 - 1/8/04	**Drilling Method:**	Washed, Driven Casing	**Core Barrel:**	NQ, 2.0 in ID, dble tube
Boring Location:	Proposed South Abutment	**Casing ID/OD:**	4 in 0-61 ft, 3 in 61+	**Water Level*:**	8.64 ft. bgs 1/6/04

Definitions:
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
60	17D	2/0	60.5 - 60.7	>100 for 2"	--	153*			59.5 No Recovery. Probable cobbles and boulders based on drilling action. Telescope to 3 inch casing at 61.0 ft.	
						NA				
	1R	42/8	62.8 - 66.3	--	--	48** Core	49.0			
						42**			62.8 Core through cobbles 62.8 - 66.3 ft. Metasiltstone and granite cobbles mixed with gravelly soils. 62.8-63.8 ft. = NA; 63.8-64.8 ft. = 2 min 28 sec; 64.8-65.8 ft. = 2 min 37 sec; 65.8-66.3 ft. = 2 min 58 sec	
						14**				
65						40**				
						82**	45.5			
						57**	44.5		66.3 Probable boulder 66.3 - 67.3 ft. based on drilling action.	
						58**	43.5		67.3 Probable cobbles 67.3 - 68.3 ft. based on drilling action.	
						82**	42.8		68.3 — 69.0 Gray, wet, very dense, SILT, some medium to fine sand, little gravel, little to trace clay (Glacial Till)	
70						83**				
						105**	40.8		71.0 Probable boulder 71.0-72.2 ft. based on drilling action.	
						102**	39.6		72.2	
	18D	16/10	73.8 - 75.1	189-56->155	>100	142**			Gray, wet, very dense, SILT, some medium to fine sand, little gravel, little to trace clay. (Glacial Till) Rock fragments in sampler tip.	
75	2R	60/18	75.4 - 80.4	--	--	Core	36.4		75.4 Core through cobbles 75.4-80.4 ft. Metasiltstone nested cobbles with rounded coarse gravel. 75.4-76.4 ft. = 3 min 0 sec; 76.4-77.4 ft. = 1 min 25 sec; 77.4-78.4 ft. = 3 min 45 sec; 78.4-79.4 ft. = 3 min 45 sec; 79.4-80.4 ft. = 5 min 40 sec	
80							31.4		80.4 **Bottom of Exploration at 80.4 feet below ground surface.**	
85										
90										

Remarks:

"*" Indicates pre-washed 4" casing blow counts
"**" Indicates pre-washed 3" casing blow counts
PPT = Pocket Penetrometer

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-OTPS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project: Irving Bridge Replacement crossing Pushaw Stream	**Location:** Route 16, Old Town, Maine	**Boring No.:**	BB-OTPS-102
		PIN:	11043.00

Driller:	Northeast Diamond Drilling	**Elevation (ft.)**	94.4	**Auger ID/OD:**	
Operator:	C. Palmer	**Datum:**	NGVD	**Sampler:**	SPT split spoon, 2 ft. long
Logged By:	R. Bennett	**Rig Type:**	CMT model 75	**Hammer Wt./Fall:**	140 lb. / 30 in.
Date Start/Finish:	1/20/04-1/21/04	**Drilling Method:**	Washed, Driven Casing	**Core Barrel:**	NQ, 2.0 in ID, dble tube
Boring Location:	Possible Center Pier Location	**Casing ID/OD:**	4.0" to 50 ft, then 3.0"	**Water Level*:**	Ice on stream at El. 102.2

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.0 - 2.0	5-8-8-7	16	17			Gray, wet, medium dense, medium to fine SAND, little gravel to 3/4 in., trace silt. (Alluvium)	A-1-b(0)/SW-SM, minus #200 = 9.4%
						25				
						27				
						30				
						25	90.4		4.0	
5	2D	24/11	5.0 - 7.0	18/8/7/8	15	40			Dark gray, wet, medium dense, coarse to fine SAND, trace fine rounded gravel. Uniformly graded. (Outwash)	A-1-b(0)/SP, minus #200 = 4.3%
						56				
						50				
						40				
						10				
10	3D	24/8	10.0 - 12.0	11/12/16/20	28	18			Dark gray, wet, medium dense, coarse to fine SAND, trace silt. (Outwash)	
						56				
						66				
						94				
						17				
15	4D	24/9	15.0 - 17.0	8/12/13/17	25	37			Dark gray, wet, medium dense, coarse to fine SAND, trace silt. (Outwash)	
						57				
						246	77.4		17.0 Increased gravel content based on drilling action	
						175				
						40				
20	5D	24/0	20.0 - 22.0	8/10/15/13	25	83			No recovery, medium dense	
						81				
						96				
						106				
						75				
25	6D	24/4	25.0 - 27.0	12/34/40/26	74	80			Gray, wet, very dense, sandy GRAVEL, little silt. (Outwash)	
						80				
						78				
						102				
						112				
30										

Remarks:

* = Indicates 3 inch OD split spoon driven with 140 lb. hammer
** = Indicates 3" casing blow counts.
Water (ice) surface of Pushaw Stream was encountered 10.3 ft. below bridge deck and 8.2 ft. above mudline on 1/21/04.
RC = Roller Cone ahead without sampling
Mudline was encountered 18.5 ft. below the top of the bridge deck.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation Soil/Rock Exploration Log US UNITS	**Project:** Irving Bridge Replacement crossing Pushaw Stream **Location:** Route 16, Old Town, Maine	**Boring No.:** BB-OTPS-102 **PIN:** 11043.00

Driller:	Northeast Diamond Drilling	**Elevation (ft.)**	94.4	**Auger ID/OD:**	
Operator:	C. Palmer	**Datum:**	NGVD	**Sampler:**	SPT split spoon, 2 ft. long
Logged By:	R. Bennett	**Rig Type:**	CMT model 75	**Hammer Wt./Fall:**	140 lb. / 30 in.
Date Start/Finish:	1/20/04-1/21/04	**Drilling Method:**	Washed, Driven Casing	**Core Barrel:**	NQ, 2.0 in ID, dble tube
Boring Location:	Possible Center Pier Location	**Casing ID/OD:**	4.0" to 50 ft, then 3.0"	**Water Level*:**	Ice on stream at El. 102.2

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	7D	24/4	30.0 - 32.0	79/51/62/32*	na*	78			Gray, wet, dense, sandy GRAVEL, little silt, 2.5 inch cobble fragment stuck in sampler drive shoe. (Outwash)	
						31				
	8D	24/7	32.0 - 34.0	32/35/38/65*	na*	71			Gray, wet, dense, gravelly, coarse to medium SAND, trace silt. (Outwash)	A-1-a(0)/SP, minus #200 = 4.6%
						39**				
						28**				
35	9D	24/5	35.0 - 37.0	12/26/22/45	48	67**	59.4		35.0 Gray, wet, dense, SAND and fine GRAVEL, little silt. (Outwash, possibly with Glacial till inclusions)	
						130**				
						150**				
						132**				
						46**				
40	10D	24/6	40.0 - 42.0	30/120/58/28	86	110**			Gray-brown, wet, very dense, sandy fine GRAVEL, little silt.	
						82**				
						177**				
45	11D	7/1	45.0 - 45.6	30/123+	na		48.8		Gray, wet, very dense, SAND and GRAVEL, little silt. (Outwash) 45.6	
						RC	48.3		Probable cobble from 45.6-46.0 ft. based on drilling action. 46.1	
									Advanced roller cone without sampling through cobbles and boulders from 46.1 - 51.5 ft.	
50										
							42.9		51.5 **Bottom of Exploration at 51.5 feet below ground surface.** Boring terminated after switching to 3 in. casing to advance through cobble/ boulder layer, and then crimping casing drive shoe.	
55										
60										

Remarks:

* = Indicates 3 inch OD split spoon driven with 140 lb. hammer
** = Indicates 3" casing blow counts.
Water (ice) surface of Pushaw Stream was encountered 10.3 ft. below bridge deck and 8.2 ft. above mudline on 1/21/04.
RC = Roller Cone ahead without sampling
Mudline was encountered 18.5 ft. below the top of the bridge deck.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project: Irving Bridge Replacement crossing Pushaw Stream
Location: Route 16, Old Town, Maine

Boring No.: BB-OTPS-103
PIN: 11043.00

Driller:	Maine DOT	**Elevation (ft.)**	112.9	**Auger ID/OD:**	
Operator:	C. MANN	**Datum:**	NGVD	**Sampler:**	SPT split spoon, 2.0 ft. long
Logged By:	R. BENNETT	**Rig Type:**	CME Model 45c	**Hammer Wt./Fall:**	140 lb. / 30 in.
Date Start/Finish:	0830 1/20/4 - 1520 1/21/4	**Drilling Method:**	Drive and Wash	**Core Barrel:**	NQ, 2.0 in. ID, dble tube
Boring Location:	Proposed North Abutment	**Casing ID/OD:**	4.0 in.	**Water Level*:**	9.62 ft. bgs 1/21/04

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows
	1D	24/8	2.5 - 4.5	65/50/33/19	83	
5	2D	24/24	5.0 - 7.0	7/2/3/3	5	
	3D	24/7	7.0 - 9.0	7/11/6/1	17	
	4D	24/12	9.0 - 11.0	WOH for 24 in.	0	
	5D	24/6	11.0 - 13.0	2/37/10/7	47	
	~~V1~~		~~11.0 - 11.3~~	~~Su=320 psf~~		
	V2		11.3 - 11.7	Could not turn ~~Su>1043 psf~~		
	6D	24/13	13.0 - 15.0	16/24/31/24	55	
20	7D	24/10	20.0 - 22.0	44/20/15/5	35	
25	8D	24/11	25.0 - 27.0	7/6/7/6	13	37

Elevation (ft.)	Visual Description and Remarks	Depth to bottom of layer (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
112.5	5 in. asphalt pavement.	0.4	
108.5	Brown, moist, very dense, coarse to fine SAND, little fine gravel, trace silt. (Road Subbase Fill)	4.4	
104.4	Olive brown, very moist, loose, silty fine SAND, trace gravel, with occasional root hairs. Brown, wet, loose to medium dense, fine sandy SILT, trace gravel to 3/4 in., trace organic root hairs and root twig.	8.5	WC = 23.0%
102.4	Brown, wet, very loose/soft, SILT and SAND, trace clay, trace root hairs, occasional black streaks/organic remnants.	10.5	
101.4	Greenish gray, wet, very soft, clayey SILT, trace fine sand, trace organics. (Glaciomarine) Olive gray, wet, soft, clayey SILT, trace fine sand, trace organics. (Glaciomarine) V1 = 20x40 vane V2 = 25.4x50.8 vane	11.5	WC = 35.0%
100.9	Olive brown, cmf SAND, trace gravel, trace silt, mixed with wood.	12.0	
100.4	6" piece of wood (well-preserved) jammed in spoon drive shoe at about 12.0 ft.	12.5	
99.9		13.0	
91.4	Gray-brown, wet, very dense, silty fine to medium SAND, some gravel to 3/4", trace organincs. (Outwash) Gray, wet, medium dense, coarse to fine SAND, some silt, little gravel. (Outwash)	21.5	
83.9	Dark gray, wet, loose to medium dense, coarse to fine SAND, trace silt. Uniform gradation. Methane odor. (Outwash)	29.0	

Casing blows (by depth): 24–25 ft: 33; 25–26 ft: 37; 26–27 ft: 38; 27–28 ft: 60; 28–29 ft: 63; 29–30 ft: 92.

Remarks:

RC = Roller Cone through rock

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project: Irving Bridge Replacement crossing Pushaw Stream		**Boring No.:**	BB-OTPS-103
Location: Route 16, Old Town, Maine		**PIN:**	11043.00
Driller:	Maine DOT	**Elevation (ft.)**	112.9
Operator:	C. MANN	**Datum:**	NGVD
Logged By:	R. BENNETT	**Rig Type:**	CME Model 45c
Date Start/Finish:	0830 1/20/4 - 1520 1/21/4	**Drilling Method:**	Drive and Wash
Boring Location:	Proposed North Abutment	**Casing ID/OD:**	4.0 in.
Auger ID/OD:		**Sampler:**	SPT split spoon, 2.0 ft. long
Hammer Wt./Fall:	140 lb. 30 in.	**Core Barrel:**	NQ, 2.0 in. ID, dble tube
Water Level*:	9.62 ft. bgs 1/21/04		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	9D	24/2	30.0 - 32.0	17/20/53/23	73	82			Gray, wet, dense to very dense, sandy GRAVEL, little silt. (Outwash)	
						115				
						87				
						95				
						111				
35	10D	24/4	35.0 - 37.0	15/6/6/8	12	67	77.9		35.0 Dark gray, wet, medium dense, medium to fine SAND, trace gravel, trace to no silt. Uniformly graded. (Outwash)	
						58				
						89				
						93				
						108				
40						90				
	11D	24/11	41.0 - 43.0	12/10/10/11	20	115			Dark gray, wet, medium dense, medium to fine SAND, trace coarse sand, trace fine gravel, trace silt. Uniformly graded. (Outwash)	
						148				
						140				
						143				
45	12D	24/7	45.0 - 47.0	13/12/18/22	30	93			Dark gray, wet, medium dense, coarse to fine SAND, little fine gravel, trace silt. (Outwash)	
						124				
						109				
						112				
						161				
50	13D	24/7	50.0 - 52.0	24/24/20/16	44	97	62.9		50.0 Gray, wet, dense, GRAVEL and coarse to fine SAND, little silt. (Outwash)	
						181				
						144				
						124				
						122				
55	14D	23/8	55.0 - 56.9	27/22/35/50+	57	146			Layer of very dark gray, wet, dense, sandy GRAVEL, little silt overlying gray, wet, dense, GRAVEL, some sand, some silt. (Outwash)	
						153				
						300+ RC	55.7		57.2 Roller coned through rock 57.2-58.7 ft.	
	R1	60/60	58.7 - 63.7	RQD = 29.1%		Core	54.2		58.7 Bedrock: Keduskeag Unit - Medium to dark grey, Poor to fair quality phyllite and metasiltstone with bands of quartzite and calcite.	
60										

Remarks:

RC = Roller Cone through rock

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project: Irving Bridge Replacement crossing Pushaw Stream		**Boring No.:**	BB-OTPS-103
Location: Route 16, Old Town, Maine		**PIN:**	11043.00

Driller:	Maine DOT	**Elevation (ft.)**	112.9	**Auger ID/OD:**	
Operator:	C. MANN	**Datum:**	NGVD	**Sampler:**	SPT split spoon, 2.0 ft. long
Logged By:	R. BENNETT	**Rig Type:**	CME Model 45c	**Hammer Wt./Fall:**	140 lb. / 30 in.
Date Start/Finish:	0830 1/20/4 - 1520 1/21/4	**Drilling Method:**	Drive and Wash	**Core Barrel:**	NQ, 2.0 in. ID, dble tube
Boring Location:	Proposed North Abutment	**Casing ID/OD:**	4.0 in.	**Water Level*:**	9.62 ft. bgs 1/21/04

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
60									Fracture density = 20 - 50+ per foot. R1: Core Times (min:sec) 58.7-59.7 (12:02) 59.7-60.7 (11:41) 60.7-61.7 (10:11) 61.7-62.7 (11:21) 62.7-63.7 (11:35) Recovery = 100%	
	R2	60/60	63.7 - 68.7	RQD = 32.5%		Core	49.2		63.7	
65									R2: Core Times (min:sec) 63.7-64.7 (10:15) 64.7-65.7 (7:25) 65.7-66.7 (7:00) 66.7-67.7 (8:30) 67.7-68.7 (7:45) Recovery = 100%	
							44.2		68.7 **Bottom of Exploration at 68.7 feet below ground surface.**	
70										
75										
80										
85										
90										

Remarks:

RC = Roller Cone through rock

APPENDIX B

LABORATORY TEST RESULTS

Geotechnical Test Report

Irving Street Bridge Replacement Project
Old Town, MA

Prepared for:

Golder Associates

Yarmouth, ME

Prepared by:

GeoTesting Express, Inc.
Boxborough, MA

January 28, 2004

Moisture Content of Soil by ASTM D 2216

Client:	Golder Associates	GTX #:	5006
Project Name:	Irving Street Bridge Replacement	Test Date:	01/27/04
Project Location:	Old Town, ME	Tested By:	njh
		Checked By:	jdt

Boring ID	Sample ID	Depth, ft	Visual Description	Moisture Content, %
---	BB-OTPS-101 S-D7	---	Moist, olive gray clay	23
---	BB-OTPS-101 S-D6	---	Moist, mottled olive and light gray clay	29
---	BB-OTPS-103 S-3s-A	---	Moist, olive clayey sand	23
---	BB-OTPS-103 S-4s-B	---	Wet, olive silt	35

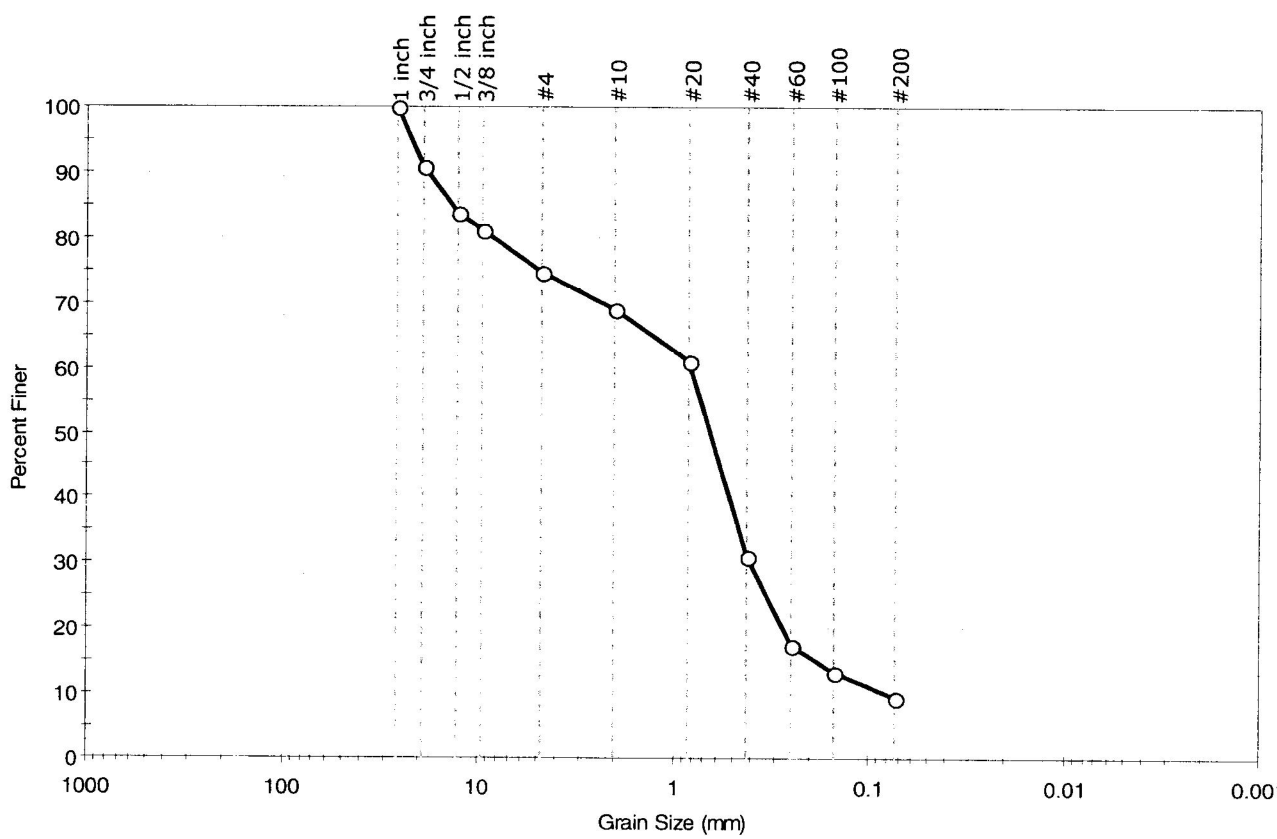
Notes:

GeoTesting Express, Inc.

Client:	Golder Associates				
Project:	Irving Bridge Replacement				
Location:	Old Town, ME			Project No:	GTX-5006
Boring ID:	---	Sample Type:	jar	Tested By:	njh
Sample ID:	BB-OTPS-102 S-1s	Test Date:	01/27/04	Checked By:	jdt
Depth :	---	Test Id:	48274		

Sample Description: Wet, olive sand with silt and gravel
Sample Comment: ---
Test Comment: ---

Particle Size Analysis -ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
	25.5	65.1	9.4

Sieve Name	Sieve Size (mm)	Percent Finer	Spec. Percent	Complies
1 inch	25.70	100		
3/4 inch	19.00	91		
1/2 inch	12.70	84		
3/8 inch	9.51	81		
#4	4.75	75		
#10	2.00	69		
#20	0.84	61		
#40	0.42	31		
#60	0.25	17		
#100	0.15	13		
#200	0.074	9		

Coefficients

D_{85} =13.6987 mm
D_{30} =0.4045 mm
D_{60} =0.8220 mm
D_{15} =0.1843 mm
D_{50} =0.6522 mm
D_{10} =0.0820 mm
C_u =10.022
C_c =0.199

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

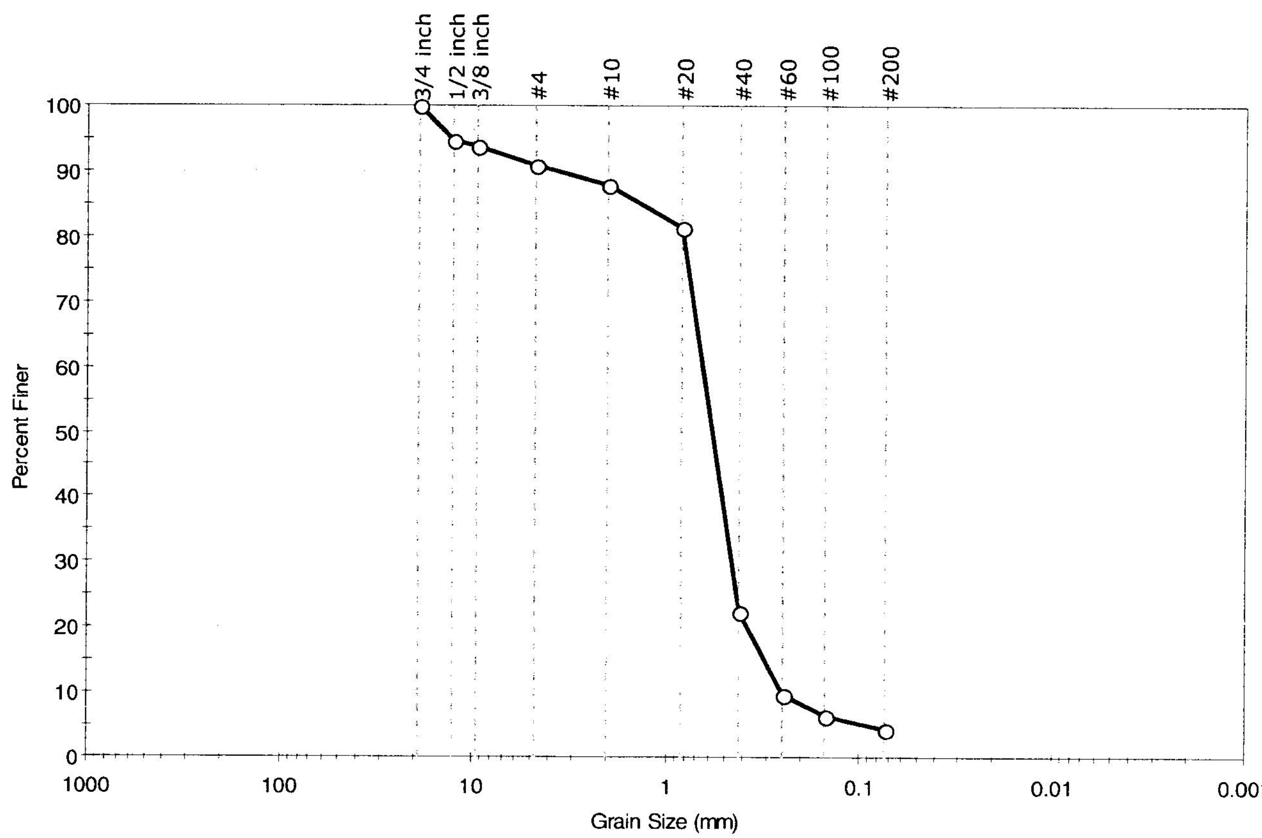
Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED
Sand/Gravel Hardness : HARD
Dispersion Device : N/A
Dispersion Period : N/A
Specific Gravity : 2.65 assumed

GeoTesting Express, Inc.

Client:	Golder Associates		
Project:	Irving Bridge Replacement		
Location:	Old Town, ME	Project No:	GTX-5006
Boring ID: ---	Sample Type: jar	Tested By:	njh
Sample ID: BB-OTPS-102 S-2s	Test Date: 01/27/04	Checked By:	jdt
Depth : ---	Test Id: 48275		
Sample Description:	Moist, dark olive gray sand		
Sample Comment:	---		
Test Comment:	---		

Particle Size Analysis -ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
	9.2	86.5	4.3

Sieve Name	Sieve Size (mm)	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.70	95		
3/8 inch	9.51	94		
#4	4.75	91		
#10	2.00	88		
#20	0.84	82		
#40	0.42	23		
#60	0.25	10		
#100	0.15	6		
#200	0.074	4		

Coefficients

D_{85} =1.3391 mm D_{30} =0.4587 mm

D_{60} =0.6527 mm D_{15} =0.3099 mm

D_{50} =0.5803 mm D_{10} =0.2532 mm

C_u =2.578 C_c =0.322

Classification

ASTM Poorly graded sand (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

Dispersion Device : N/A

Dispersion Period : N/A

Specific Gravity : 2.65 assumed

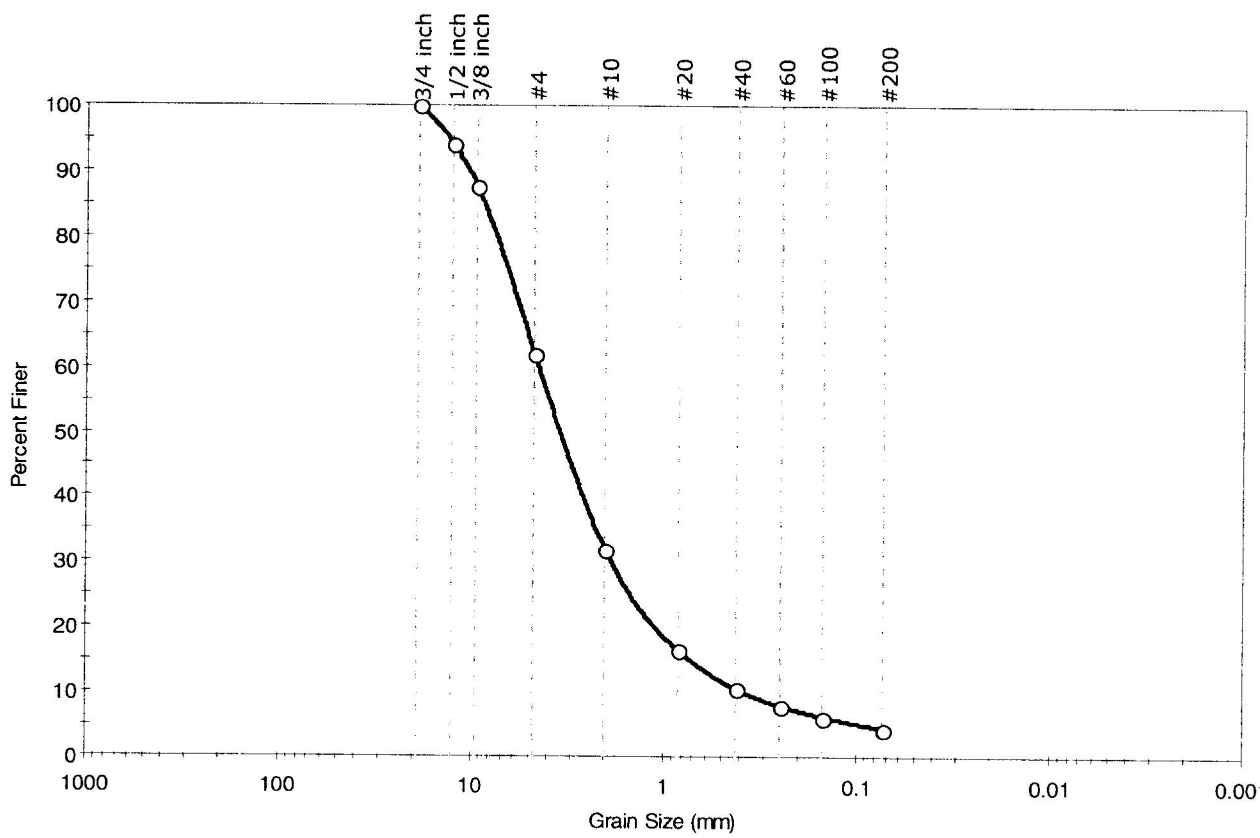
GeoTesting Express, Inc.

Client:	Golder Associates		
Project:	Irving Bridge Replacement		
Location:	Old Town, ME	Project No:	GTX-5006

Boring ID:	---	Sample Type:	jar	Tested By:	njh
Sample ID:	BB-OTPS-102 S-8s	Test Date:	01/27/04	Checked By:	jdt
Depth :	---	Test Id:	48276		

Sample Description:	Wet, olive sand with gravel
Sample Comment:	One unrepresentative ~1.5 inch rock removed from sample.
Test Comment:	---

Particle Size Analysis -ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
	38.0	57.5	4.6

Sieve Name	Sieve Size (mm)	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.70	94		
3/8 inch	9.51	88		
#4	4.75	62		
#10	2.00	32		
#20	0.84	17		
#40	0.42	11		
#60	0.25	8		
#100	0.15	6		
#200	0.074	5		

Coefficients

D_{85} =8.8502 mm	D_{30} =1.8103 mm
D_{60} =4.4856 mm	D_{15} =0.6990 mm
D_{50} =3.3705 mm	D_{10} =0.3756 mm
C_u =11.941	C_c =0.731

Classification

ASTM Poorly graded sand with gravel (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : N/A

Dispersion Period : N/A

Specific Gravity : 2.65 assumed

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APPENDIX C

CALCULATIONS

Golder Associates Manchester, New Hampshire	SUBJECT Irving Bridge Replacement		
	Job No. 043-6311	Made By: MSC	Date: 7-8-05
		Checked: pec	
	Ref.	Reviewed:	Sheet 1 of 1

EARTH PRESSURES

FOR CAST-IN-PLACE INTEGRAL ABUTMENTS, PASSIVE EARTH PRESSURES SHOULD BE USED FOR DESIGN LOADING. THE PASSIVE EARTH PRESSURE COEFFICIENT, K_p, SHOULD BE CALCULATED PER FOLLOWING EQUATION (REF. - MDOT BDG, SECTION 3.6.6, Pg. 3-8):

$$K_p = \frac{\sin(\alpha-\phi)^2}{\sin\alpha^2 \cdot \sin(\alpha+\delta) \cdot \left[1-\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi+\beta)}{\sin(\alpha+\delta)\cdot\sin(\alpha+\beta)}}\right]^2}$$

WHERE:

α = ANGLE (DEG.) OF BACK OF WALL TO HORIZ. (BDG FIG 3-1)

ϕ = BACKFILL FRICTION ANGLE

δ = INTERFACE FRICTION - SOIL / CONCRETE = $2/3\,\phi$

β = SLOPE OF BACKFILL GROUND SURFACE (DEG.)

CONSIDER TWO BACKFILL SOIL TYPES -

1. GRANULAR BORROW UNDERWATER BACKFILL (MDOT 703.19), BDG TYPE 4 SOIL (pg 3-3)
2. GRAVEL BORROW (MDOT 703.20), BDG TYPE 5 SOILS (pg. 3-3)

DESCRIPTION	SOIL TYPE	ϕ *	δ	α	β	CALCULATED K_p
GRANULAR BORROW UNDERWATER BACKFILL	4	32°	21.3°	90°	0°	7.37
GRAVEL BORROW	5	36°	24°	90°	0°	11.13

* REF: BDG, TABLE 3-3, PAGE 3-3

OBJECTIVE

To determine the allowable axial pile capacity (Q_{all}) for pile foundations at the abutments and center piers of the proposed replacement Irving Bridge.

METHOD

1. Determine the allowable structural capacity from Table 5-6 of the Maine Department of Transportation *Bridge Design Guide* (Reference 1).
2. Determine the allowable geotechnical capacity using Nordlund's Method (Reference 2).

ASSUMPTIONS

1. Frictional resistance disturbance is neglected in near-surface soil strata due to possible construction disturbance. This zone is assumed to extend about 15 ft below ground surface (bgs) at abutments and 7 ft below existing mudline at the center pier.
2. Overburden stresses from near surface soil strata are applied and considered in calculation of friction resistance from lower soil strata. This assumes scour does not occur at abutments. An assumed scour depth of 4 ft. is assumed at center pier and this weight of overburden is not included in calculation of frictional resistance.
3. Approximate maximum span length of 150 ft. Abutment type unknown.
4. No downdrag effects due to loads. Surficial soil compressible soils assumed removed during construction.

REFERENCES

1. *Bridge Design Guide.* Maine Department of Transportation. August, 2003.
2. *Design and Construction of Driven Pile Foundations.* FHWA Pub. No. FHWA-HI-97-013. November, 1998.
3. Das, Braja M. *Principals of Foundation Engineering, fourth edition.* PWS Publishing,1999.

CALCULATIONS

1. The allowable structural capacity ($Q_{all,st}$)of selected H-pile sections was determined from Reference 1, according to the following assumptions:
   a. Use of 50 ksi steel for difficult driving conditions (Reference 1, page 6.26)
   b. Use of FS = 4.0 for friction piles and all integral abutment piles (Reference 1, page 6.26)
   c. Allowable structural capacities of various H-pile sections, as presented in Table A, taken from Table 5-6 (Reference 1)

TABLE A: ALLOWABLE AXIAL STRUCTURAL PILE CAPACITY

Pile Section	Allowable Axial Structural Capacity (kip)
HP 14x73	268
HP 14x89	326
HP 14x102	375
HP 14x117	430

2. Determine allowable geotechnical capacity based on Nordlund Method as presented in Reference 2 and Attachment D.
   a. Ultimate Geotechnical capacity is based on soil cross section presented in Attachment A and the following assumptions:
      i. No bedrock was encountered during field exploration for the south abutment and center pier locations. Piles at these two locations are assumed to derive capacity from shaft friction and end bearing resistance.
      ii. Bedrock was encountered at 57 ft-bgs at the proposed location of the north abutment. For piles at this location, the Nordlund method as presented in Reference 2 (Attachment D) is used for cases where pile depth does not reach bedrock. If piles are driven to bedrock at this location, it is assumed they will be driven to refusal, and that all of the pile geotechnical capacity will be derived from end bearing resistance.
      iii. Depth numbers shown in parenthesis in Attachment A are for depth below existing grade or mudline. For the purposes of this calculation, an additional fill depth of 1.0 and 2.5 feet will be added to the south and north abutment locations, respectively, to reflect final grades. At the center pier a scour depth of 4 ft. is assumed for the calculations. Actual scour depth estimates will be determined by MaineDOT and could vary from the assumed 4 ft. value.
      iv. Friction disturbance is neglected in near surface strata due to possible construction disturbance. This zone is assumed to extend about 15 ft below ground surface (ft-bgs) at the abutments and 7 ft below existing mudline at the center pier. Overburden stresses from near surface soil strata are applied and considered in calculation of friction resistance from lower soil strata. This assumes scour does not occur at abutments. An assumed scour depth of 4 ft. is assumed at center pier and this weight of overburden is not included in calculation of frictional resistance.
      v. The effective pile perimeter will be the box perimeter, not the H-pile perimeter.
      vi. The effective pile tip area used was the H-pile area, not the box area.
      vii. This analysis did not consider the effects of negative skin friction due to settlement.
      viii. For preliminary design purposes, a 14 x 89 pile section was selected.
   b. Ultimate geotechnical capacity was calculated based on Nordlund Method as presented in Reference 2 and Attachment D. Total ultimate geotechnical capacity ($Q_{ult,geo}$) was calculated as the sum of resistance from tip (R_t) and shaft (R_s) as:

$$Q_{ult,geo} = R_s + R_t$$

   If the pile tip is supported on bedrock, R_x is assumed to be zero.
      i. Tip resistance (R_t) was calculated based on the following equation:

$$R_t = A_t q' \alpha N_q^* \leq q_L A_t$$

      where:

A_t	=	Tip area of the pile considered.
q'	=	Effective overburden pressure at pile tip.
α	=	Dimensionless factor from Figure 9-16a, Reference 2.
N_q^*	=	Bearing capacity factor from Figure 9-16b, Reference 2.
q_L	=	Limiting unit toe resistance from Figure 9.17, Reference 2.

      ii. Shaft resistance (R_s) was calculated based on

$$R_s = \sum K_\delta C_f P_d \sin(\delta) C_D D$$

where:

K_δ = Coefficient of lateral stress at depth. See Tables 9-2a and 9-2b, Reference 2.

C_f = Correction factor when $\delta \neq \phi$. See Figure (9.15), Reference 2.

P_d = Average effective overburden pressure for the soil layer (assumed to be effective overburden pressure at midpoint of soil layer).

δ = Pile-soil friction angle, a function of ϕ and the pile's specific volume (V). See Figure (9.10), Reference 2.

C_D = Effective pile perimeter

D = Length of pile segment considered

c. Allowable geotechnical capacity ($Q_{all,geo}$) was calculated from the ultimate geotechnical capacity ($Q_{ult,geo}$) using a factor of safety of 2.25. This factor of safety was determined from Table 5-7 (Reference 1) according to the assumption that dynamic testing and wave equation analysis will be performed to verify static capacity calculations. Allowable geotechnical capacity was calculated according to the following equation:

$$Q_{all,geo} = \frac{Q_{ult,geo}}{FS}$$

SAMPLE CALCULATION: For an HP 14x89 pile installed to 29.00 ft-bgs at the location of the south abutment.

1. Allowable structural axial capacity determined for an HP 14x89 pile from Reference 1, shown in Table A. Structural axial capacity will govern for piles installed in rock.
2. Allowable geotechnical capacity calculated based on Nordlund Method as presented in Reference 2 and Attachment D.
   a. Total ultimate geotechnical capacity ($Q_{ult,geo}$) was calculated as the sum of resistance from tip (R_t) and shaft (R_s) as:

$$Q_{ult,geo} = R_s + R_t$$

i. Tip resistance (R_t) was calculated based on the following equation:

$$R_t = A_t q' \alpha N_q^* \leq q_L A_t$$

For a pile tip located at 29.00 ft-bgs at the location of the south abutment:

A_t = Tip area of the pile = 26.1 in^2 = 0.181 ft^2 (Attachment B)

q' = Effective overburden pressure at pile tip

$$q' = (5ft*135pcf) + (9ft - 5ft)*(125pcf) + (13ft - 9ft)*(125pcf - 62.4pcf) + \ldots$$
$$\ldots + (15.5ft - 13ft)*(115pcf - 62.4pcf) + (29ft - 15.5ft)*(125pcf - 62.4pcf) = \ldots$$
$$\ldots = (5ft*135pcf) + (4ft)*(125pcf) + (4ft)*(62.6pcf) + (2.5ft)*(52.6pcf) + \ldots$$
$$\ldots + (13.5ft)*(62.6pcf) = 2402psf$$

α = Dimensionless factor from Reference 2, Figure 9-16a. = 0.63 (for D/b = 25, ϕ = 33°)

N_q^* = Bearing capacity factor from Reference 2, Figure 9-16b. = 45 (for ϕ = 33°)

q_L = Limiting unit toe resistance from Reference 2, Figure 9.17

$$= 2000 \text{ kPa} * \frac{1000Pa}{kPa} * \frac{psi}{6894Pa} * \frac{144in^2}{ft^2} = 41775 \text{ psf}$$

According to these figures:

$R_t = A_t q' \alpha N_q^* \le q_L A_t$ = 0.181 ft²* 2402psf *0.67*43<=41775psf*0.181 ft²
=12.52 kip >= **7.57 kip**. *Rt is limited by 7.57 kip.*

ii. For a pile driven to a depth of 29 ft bgs, shaft resistance (R_s) was calculated based on

$$R_s = K_\delta C_f P_d \sin(\delta) C_D D$$

where:

K_δ = 1.03 + [(.0168 - .0093)/(.0186 - .0093) x (1.17 – 1.03)] = 1.14
Coefficient of lateral stress at depth. See Tables (9-2a and 9-2b – Reference 2). (For pile displacement volume of 0.168 use linear interpolation of K_δ values from those given for V = 0.0093 and 0.0186.)

C_f = 0.93 = Correction factor when $\delta \neq \phi$. See Figure (9.15)

P_d = Average effective overburden pressure for the soil layer. This was calculated as the effective overburden pressure at midpoint of soil layer, or the average of overburden pressure values at the top and bottom of the layer:

$$= \frac{\sigma_{z=15.51ft} + \sigma_{z=29.00ft}}{2} = \frac{1558psf + 2402psf}{2} = 1980psf$$

δ = Pile-soil friction angle, a function of ϕ and the pile's specific volume (V). From Figure (9.10), $\frac{\delta}{\phi} = 0.8 \therefore \delta = 0.8\phi = 0.8 * 33^\circ = 26.4^\circ$

C_D = Effective pile perimeter = Box pile perimeter = 2d +2b = (2*13.83in+2*14.695in)*(1ft/12in) = 4.75 ft from Attachment B.

D = Length of pile segment considered = 29 ft -15.5 ft = 13.5 ft

Using the above figures:

$$R_s = K_\delta C_f P_d \sin(\delta) C_D D = 1.14 * 0.93 * 1980pcf * \sin(26.4) * 4.75ft * \ldots$$

$$\ldots * 13.5ft * \frac{1kip}{1000lb} = 59.9kip$$

iii. Total ultimate geotechnical capacity for a pile driven to 29 ft-bgs was calculated according to:

$$Q_u = R_s + R_t = 7.6 \text{ kip} + 59.9 \text{ kip} = \mathbf{67.5\ kip}$$

b. Allowable geotechnical capacity ($Q_{all,geo}$) was calculated from the ultimate geotechnical capacity ($Q_{ult,geo}$) using a factor of safety of 2.25. This factor of safety was determined from Table 5-7 (Reference 1) according to the assumption that dynamic testing and wave equation analysis will be performed to verify static capacity calculations. Ultimate geotechnical capacity was calculated according to the following equation:

$$Q_{all,geo} = \frac{Q_{ult,geo}}{FS} = \frac{67.5kip}{2.25} = 30.0kip$$

CONCLUSION:

A summary of pile capacity calculations for various pile lengths at the Irving Bridge north and south abutments and center pier locations are presented in Attachment C.

A summary of the allowable geotechnical pile capacity for various pile lengths at the proposed south abutment, center pier and north abutment is presented on Table B and Figure A. Due to deep bedrock observed in borings, it is expected that all piles (with the possible exception of piles in the area of the proposed north abutment) will be installed as friction piles. Due to difficult driving conditions observed in borings, pile driving is expected to be more difficult with increasing depth. Given these design considerations, Table B presents a summary of pile capacities for each location based on probability of driving depth below finished grade.. Note that for the case of pile driven to bedrock for the north abutment, pile capacity is governed by the structural capacity of the pile, not by geotechnical capacity.

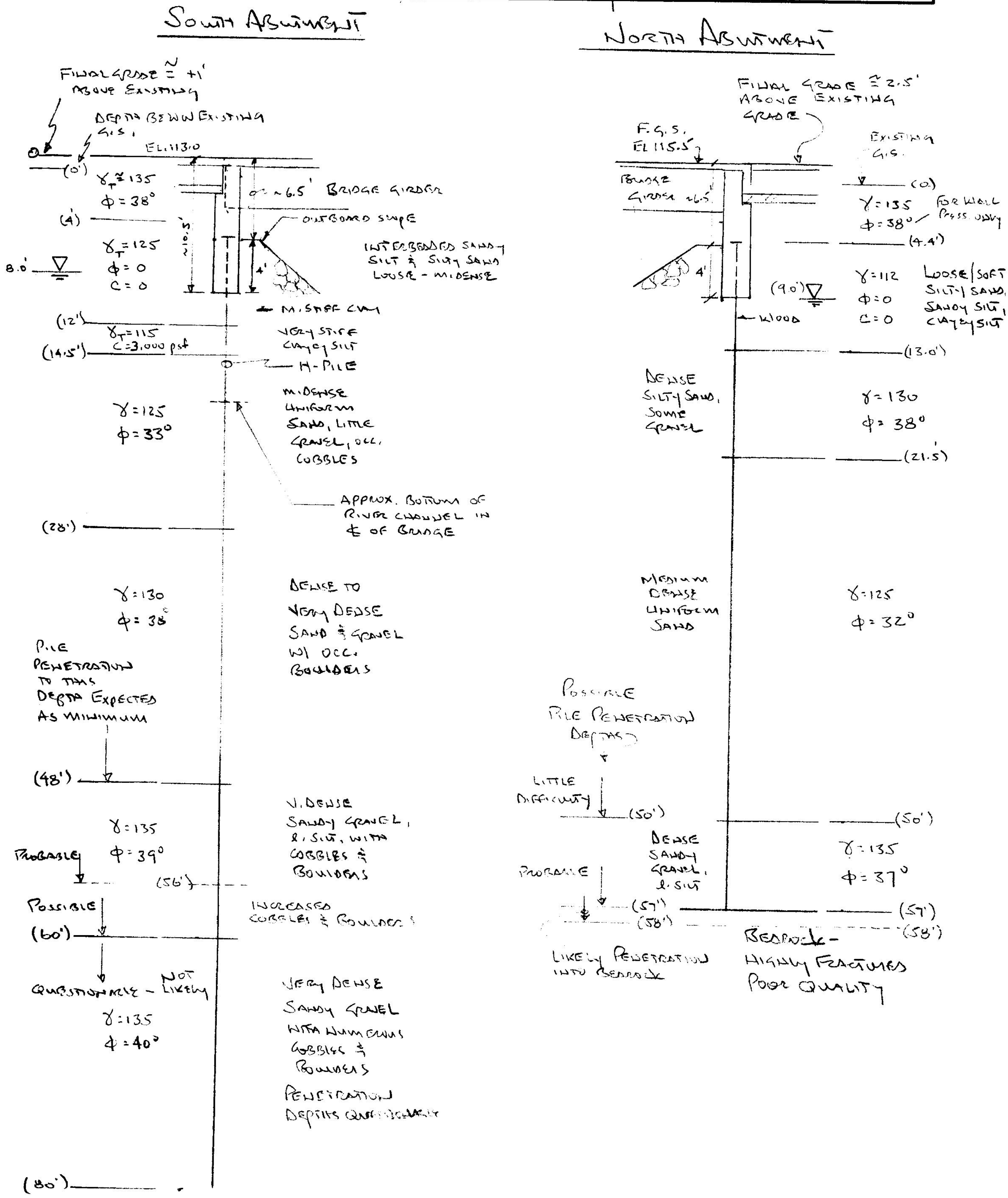
TABLE B:
PILE CAPACITY BASED ON DRIVING DEPTH PROBABILITY
IRVING BRIDGE REPLACEMENT
ROUTE 16, OLD TOWN MAINE

Location	Probability of Achieving Driving Depth	Driving Depth (ft)	Tip Elevation (ft-msl)	Q_{all} (kip)
South Abutment	Probable	49	64	149
	Possible	61	52	247
	Not Likely	>61	<52	>247
Center Pier	Probable	42	48.4	125
	Not Likely	>46	<44	>160
North Abutment	Probable	52.5	63	112
	Possible	59.5	56	172
	Possibly Driven to Bedrock	60.5	55	326

ATTACHMENT A TO APPENDIX C

Golder Associates
Manchester, New Hampshire

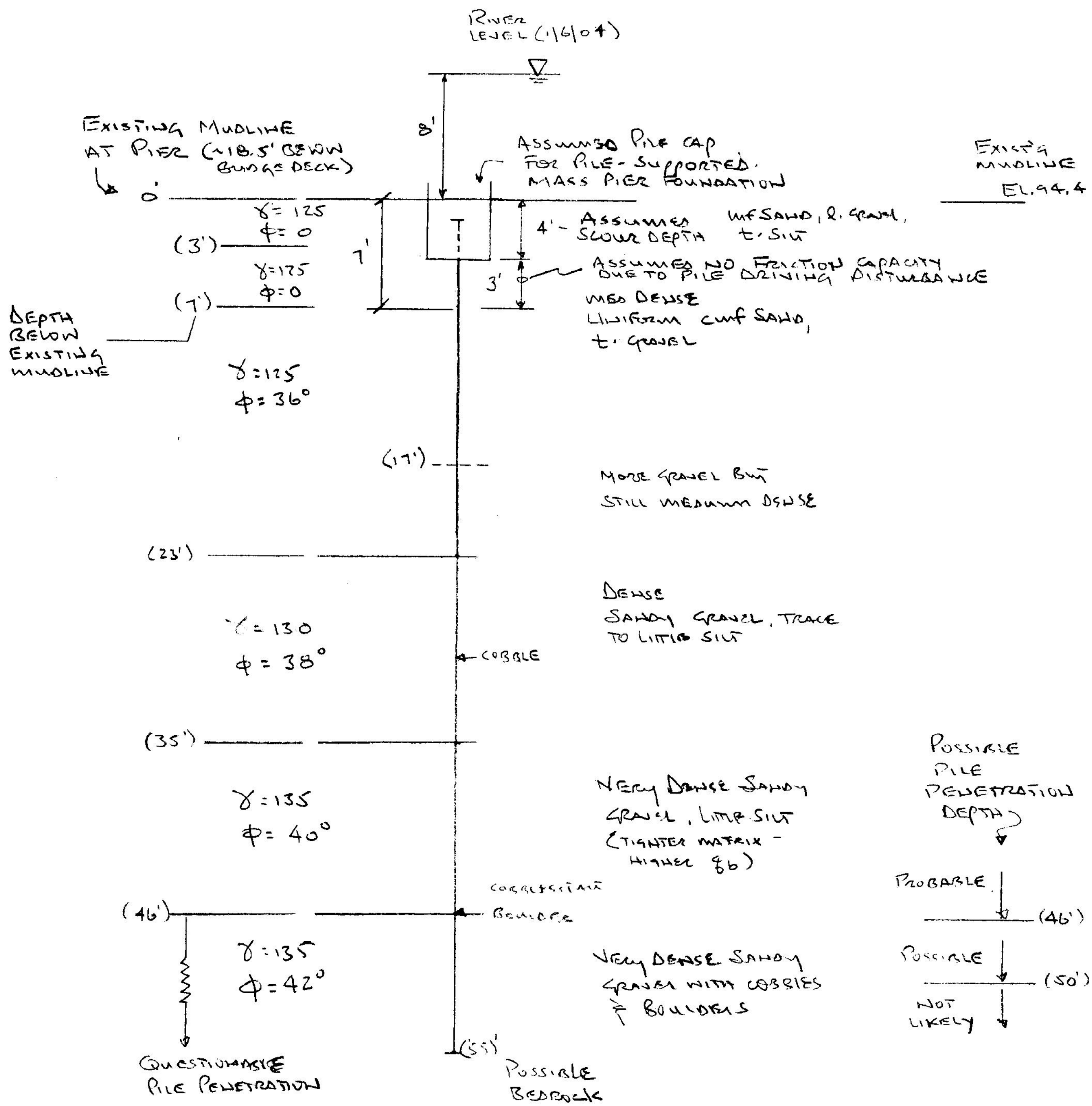
SUBJECT REC'D SOIL PARAMETERS FOR STATIC PILE CAPACITY ANALYSIS - IRVING BRIDGE		
Job No. 043-6811	Made By: MSP	Date: 8-19-05
	Checked:	
Ref.	Reviewed: pcc	Sheet 1 of 2



Golder Associates
Manchester, New Hampshire

SUBJECT	Soil Parameters For Pile Analysis		
Job No. 043-6311	Made By: MSP	Date: 8-19-05	
	Checked:		
Ref.	Reviewed: pcc	Sheet 2 of 2	

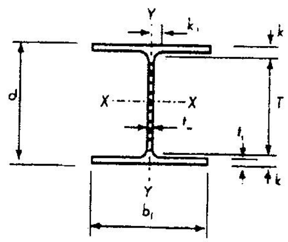
Center Pier



Attachment B
1/1

H-Pile Shapes

Dimensions and Properties



ASTM A572 Gr. 50 and 60
ASTM A588
ASTM A690
Inquire for other grades.

U.S. STANDARD																
Designation	Area	Depth	Web Thickness		Flange		Distance		Elastic Properties						Surface Area Per Linear ft.
					Width	Thick			X-X Axis			Y-Y Axis			
	A	d	t_w	$t_w/2$	b_f	t_f	k	k_1	I_x	S_x	r_x	I_y	S_y	r_y	
	in.^{2}	in.	in.	in.	in.	in.	in.	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	ft^2/ft
HP14 x 117	34.4	14.21	0.805	0.403	14.885	0.805	1.5000	1.0625	1220	172	5.96	443	59.5	3.59	7.10
x 102	30.0	14.01	0.705	0.353	14.785	0.705	1.3750	1.0000	1050	150	5.92	380	51.4	3.56	7.05
x 89	26.1	13.83	0.615	0.308	14.695	0.615	1.3125	0.9375	904	131	5.88	326	44.3	3.53	7.00
x 73	21.4	13.61	0.505	0.253	14.585	0.505	1.1875	0.8750	729	107	5.84	261	35.8	3.49	6.95
HP12 x 84	24.6	12.28	0.685	0.343	12.295	0.685	1.3750	1.0000	650	106	5.14	213	34.6	2.94	5.93
x 74	21.8	12.13	0.605	0.303	12.215	0.610	1.3125	0.9375	569	93.8	5.11	186	30.4	2.92	5.89
x 63	18.4	11.94	0.515	0.258	12.125	0.515	1.2500	0.8750	472	79.1	5.06	153	25.3	2.88	5.85
x 53	15.5	11.78	0.435	0.218	12.045	0.435	1.1250	0.8750	393	66.8	5.03	127	21.1	2.86	5.81
HP10 x 57	16.8	9.99	0.565	0.283	10.225	0.565	1.1875	0.8125	294	58.8	4.18	101	19.7	2.45	4.89
x 42	12.4	9.70	0.415	0.208	10.075	0.420	1.0625	0.7500	210	43.4	4.13	71.7	14.2	2.41	4.82
HP8 x 36	10.6	8.02	0.445	0.223	8.1	0.445	0.9375	0.6250	119	29.8	3.36	40.3	9.88	1.95	3.89

Foster Piling

Pile Properties:

Type = 14x89
At = 26.1 in^2 = tip area of pile
d= 13.83
b= 14.7 in = width of pile
Pile specifics are from Attachment B

Soil Properties

Dw = 9 ft = depth to gw table (future).
See Attachment A for soil profiles. Existing grade is ~1.0 ft below assumed future grade. Assume no friction resistance to 15.5 ft-bgs (future).

R_t Tip capacity

$$R_t = A_t q' \alpha N_q^* \le q_L A_t$$

use b= 13.83 = pile "diameter" (Attachment B)

Depth	gamma'	PHI	At	D/b	Alpha	Nq'	q'	Qp	q-limit	q-limit	Qp limit	Qp-ult
ft	pcf	deg.	ft^2				psf	kip	kPa	psf	kip	kip
0.01	135	0	0.181	0.0087	0	0	1.35	0	0	0	0	0
5	135	0	0.181	4.3384	0	0	675	0	0	0	0	0
5.01	125	0	0.181	4.3471	0	0	676.25	0				
9	125	0	0.181	7.8091	0	0	1175	0	0	0	0	0
9.01	62.6	0	0.181	7.8178	0	0	1175.6	0				
13	62.6	0	0.181	11.28	0	0	1425.4	0				
13.01	52.6	0	0.181	11.289	0	0	1425.9	0	0	0	0	0
15.5	52.6	0	0.181	13.449	0	0	1556.9	0				
15.51	62.6	33	0.181	13.458	0.64	45	1557.5	8.1303	2000	41775.46	7.571802	7.571802
29	62.6	33	0.181	25.163	0.61	45	2402	11.951	2000	41775.46	7.571802	7.571802
29.01	67.6	38	0.181	25.171	0.71	110	2402.7	34.011	12000	250652.7	45.43081	34.01138
49	67.6	38	0.181	42.516	0.71	110	3754	53.14	12000	250652.7	45.43081	45.43081
49.01	72.6	39	0.181	42.525	0.72	120	3754.7	58.799	15000	313315.9	56.78851	56.78851
61	72.6	39	0.181	52.928	0.72	120	4625.2	72.431	15000	313315.9	56.78851	56.78851
61.01	72.6	40	0.181	52.937	0.73	150	4625.9	91.81	20000	417754.6	75.71802	75.71802
70	72.6	40	0.181	60.738	0.73	150	5278.6	104.76	20000	417754.6	75.71802	75.71802

R_s Shaft resistance

$$R_s = \sum K_\delta C_f P_d \sin(\delta) C_D D$$

delta/phi= 0.8 from Figure 9.10, Reference 2.
d= 13.83 in = pile depth
b= 14.7 in = pile flange depth
Cd= 4.755 ft = (2d+2b)/12
V= 0.0168 m^2, converted from Attachment B

Depth	gamma'	PHI	Cd	Pd Avg		delta	Kd	Cf	dZ	Qsi	Qs	Qs
ft	pcf	deg.	ft	psf					ft	lb	lb	kip
0.01	135	0	4.76	1.35		0.00	0	0	0.01	0	0	0
5	135	0	4.76	675		0.00	0	0	4.99	0	0	0
5.01	125	0	4.76	676.3		0.00	0	0	0.01	0	0	0
9	125	0	4.76	1175		0.00	0	0	3.99	0	0	0
9.01	62.6	0	4.76	1176		0.00	0	0	0.01	0	0	0
13	62.6	0	4.76	1425		0.00	0	0	3.99	0	0	0
13.01	52.6	0	4.76	1426		0.00	0	0	0.01	0	0	0
15.5	52.6	0	4.76	1557		0.00	0	0	2.49	0	0	0
15.51	62.6	33	4.76	1980		26.40	1.14	0.93	0.01	44.3767	44.3767	0.04438
29	62.6	33	4.76	1980		26.40	1.14	0.93	13.49	59864.2	59908.6	59.9086
29.01	67.6	38	4.76	3078		30.40	1.73	0.9	0.01	115.328	60023.9	60.0239
49	67.6	38	4.76	3078		30.40	1.73	0.9	19.99	230541	290565	290.565
49.01	72.6	39	4.76	4190		31.20	1.87	0.9	0.01	173.699	290738	290.738
61	72.6	39	4.76	4190		31.20	1.87	0.9	11.99	208265	499003	499.003
61.01	72.6	40	4.76	4952		32.00	2.01	0.88	0.01	220.72	499224	499.224
70	72.6	40	4.76	4952		32.00	2.01	0.88	8.99	198428	697652	697.652

Summary of Results

Geotechnical Pile Capacity
Irving Bridge, South Abutment

Existing ~112.0 | New Fill +1.0

Elev	Depth	Rt	Rs	Qu	Qall South Abutment
0=113.0	*(ft)*	*(kip)*			
113.0	0.01	0.0	0.0	0.0	0.0
108.0	5	0.0	0.0	0.0	0.0
108.0	5.01	0.0	0.0	0.0	0.0
104.0	9	0.0	0.0	0.0	0.0
104.0	9.01	0.0	0.0	0.0	0.0
100.0	13	0.0	0.0	0.0	0.0
100.0	13.01	0.0	0.0	0.0	0.0
97.5	15.5	0.0	0.0	0.0	0.0
97.5	15.51	7.6	0.0	7.6	3.4
84.0	29	7.6	59.9	67.5	30.0
84.0	29.01	34.0	60.0	94.0	41.8
64.0	49	45.4	290.6	336.0	149.3
64.0	49.01	56.8	290.7	347.5	154.5
52.0	61	56.8	499.0	555.8	247.0
52.0	61.01	75.7	499.2	574.9	255.5
43.0	70	75.7	697.7	773.4	343.7

Pile Properties:

Type = 14x89
At = 26.1 in^2 = tip area of pile
d= 13.83
b= 14.7 in = width of pile
Pile specifics are from Attachment B

Soil Properties

Dw = 11.5 ft = depth to gw table (future)
See Attachment A for soil profiles. Existing grade is ~2.5 ft below assumed future grade. Assume no friction resistance to 15.5 ft-bgs (future).

R_t Tip capacity

$$R_t = A_t q' \alpha N_q^* \leq q_L A_t$$

use b = 13.83 = pile "diameter" (Attachment B)

Depth	gamma'	PHI	At	D/b	Alpha	Nq'	q'	Qp	q-limit	q-limit	Qp limit	Qp-ult
ft	pcf	deg.	ft^2				psf	kip	kPa	psf	kip	kip
0.01	135	0	0.181	0.00868	0	0	1.35	0	0	0	0	0
6.9	135	0	0.181	5.98698	0	0	931.5	0	0	0	0	0
6.91	112	0	0.181	5.99566	0	0	932.6	0				
11.5	112	0	0.181	9.97831	0	0	1447	0	0	0	0	0
11.51	49.6	0	0.181	9.98698	0	0	1447	0				
15.5	49.6	0	0.181	13.449	0	0	1645	0				
15.51	67.6	38	0.181	13.4577	0.71	110	1646	23.3	12000	250653	45.4308	23.297
24	67.6	38	0.181	20.8243	0.71	110	2220	31.42	12000	250653	45.4308	31.4212
24.01	62.6	32	0.181	20.833	0.58	40	2220	9.336	1200	25065.3	4.54308	4.54308
52.5	62.6	32	0.181	45.5531	0.58	40	4004	16.84	1200	25065.3	4.54308	4.54308
52.51	72.6	37	0.181	45.5618	0.7	90	4005	45.73	10000	208877	37.859	37.859
59.5	72.6	37	0.181	51.6269	0.7	90	4512	51.52	10000	208877	37.859	37.859

R_s Shaft resistance

$$R_s = \sum K_\delta C_f P_d \sin(\delta) C_D D$$

delta/phi= 0.8 from Figure 9.10, Reference 2.
d= 13.83 in = pile depth
b= 14.7 in = pile flange depth
Cd= 4.755 ft = (2d+2b)/12
V= 0.0168 m^2, converted from Attachment B

Depth	gamma'	PHI	Cd	Pd Avg		delta	Kd	Cf	dZ	Qsi	Qs	Qs
ft	pcf	deg.	ft	psf					ft	lb	lb	kip
0.01	135	0	4.76	1.35		0.00	0	0	0.01	0	0	0
6.9	135	0	4.76	931.5		0.00	0	0	6.89	0	0	0
6.91	112	0	4.76	932.62		0.00	0	0	0.01	0	0	0
11.5	112	0	4.76	1446.7		0.00	0	0	4.59	0	0	0
11.51	49.6	0	4.76	1447.2		0.00	0	0	0.01	0	0	0
15.5	49.6	0	4.76	1645.1		0.00	0	0	3.99	0	0	0
15.51	67.6	38	4.76	1932.74		30.86	1.73	0.9	0.01	73.3888	73.3888	0.07339
24	67.6	38	4.76	1932.74		30.86	1.73	0.9	8.49	62307.1	62380.5	62.3805
24.01	62.6	32	4.76	3112.06		25.98	1.07	0.94	0.01	65.2084	62445.7	62.4457
52.5	62.6	32	4.76	3112.06		25.98	1.07	0.94	28.49	185779	248224	248.224
52.51	72.6	37	4.76	4258.26		30.04	1.58	0.9	0.01	144.155	248369	248.369
59.5	72.6	37	4.76	4258.26		30.04	1.58	0.9	6.99	100764	349133	349.133

Summary of Results

Geotechnical Pile Capacity
Irving Bridge, South Abutment

Existing ~113.0 | New Fill 2.5

Elev	Depth	Rt	Rs	Qu	Qall North Abutment
0=115.5	(ft)	(kip)			
115.5	0.01	0.0	0.0	0.0	0.0
108.6	6.9	0.0	0.0	0.0	0.0
108.6	6.91	0.0	0.0	0.0	0.0
104.0	11.5	0.0	0.0	0.0	0.0
104.0	11.51	0.0	0.0	0.0	0.0
100.0	15.5	0.0	0.0	0.0	0.0
100.0	15.51	23.3	0.1	23.4	10.4
91.5	24	31.4	62.4	93.8	41.7
91.5	24.01	4.5	62.4	67.0	29.8
63.0	52.5	4.5	248.2	252.8	112.3
63.0	52.51	37.9	248.4	286.2	127.2
56.0	59.5	37.9	349.1	387.0	172.0
55.0	60.5				326.0

Pile Properties:

Type = 14x89
At = 26.1 in^2 = tip area of pile
d= 13.83
b= 14.7 in = width of pile
Pile specifics are from Attachment B

Soil Properties

See Attachment A for soil profiles. Depths are below existing mudline in river minus assumed scour depth (4 ft.) Assume no friction resistance to elev. 87.0 ft (7 ft. below existing mudline).

R_t Tip capacity

$$R_t = A_t q' \alpha N_q^* \le q_L A_t$$

use b = 13.83 = pile "diameter" (Attachment B)

Depth	gamma	PHI	At	D/b	Alpha	Nq'	q'	Qp	q-limit	q-limit	Qp limit	Qp-ult
ft	pcf	deg.	ft^2				psf	kip	kPa	psf	kip	kip
0.01	125	0	0.181	0.0087	0	0	1.25	0	0	0	0	0
3	125	0	0.181	2.603	0	0	188.4	0	0	0	0	0
3.01	125	36	0.181	2.6117	0.69	75	189.1	1.773	8000	167102	30.2872	0
19	125	36	0.181	16.486	0.69	75	1190	11.16	8000	167102	30.2872	11.1621
19.01	130	38	0.181	16.495	0.71	110	1191	16.86	12000	250653	45.4308	16.8551
31	130	38	0.181	26.898	0.71	110	2001	28.33	12000	250653	45.4308	28.3286
31.01	135	40	0.181	26.907	0.74	150	2002	40.28	20000	417755	75.718	40.2767
42	135	40	0.181	36.443	0.74	150	2800	56.33	20000	417755	75.718	56.329
42.01	135	42	0.181	36.451	0.77	230	2801	89.9	28000	584856	106.005	89.8959
51	135	42	0.181	44.252	0.77	230	3453	110.8	28000	584856	106.005	106.005

R_s Shaft resistance

$$R_s = \sum K_\delta C_f P_d \sin(\delta) C_D D$$

delta/phi= 0.8 from Figure 9.10, Reference 2.
d= 13.83 in = pile depth
b= 14.7 in = pile flange depth
Cd= 4.755 ft = (2d+2b)/12
V= 0.0168 m^2, converted from Attachment B

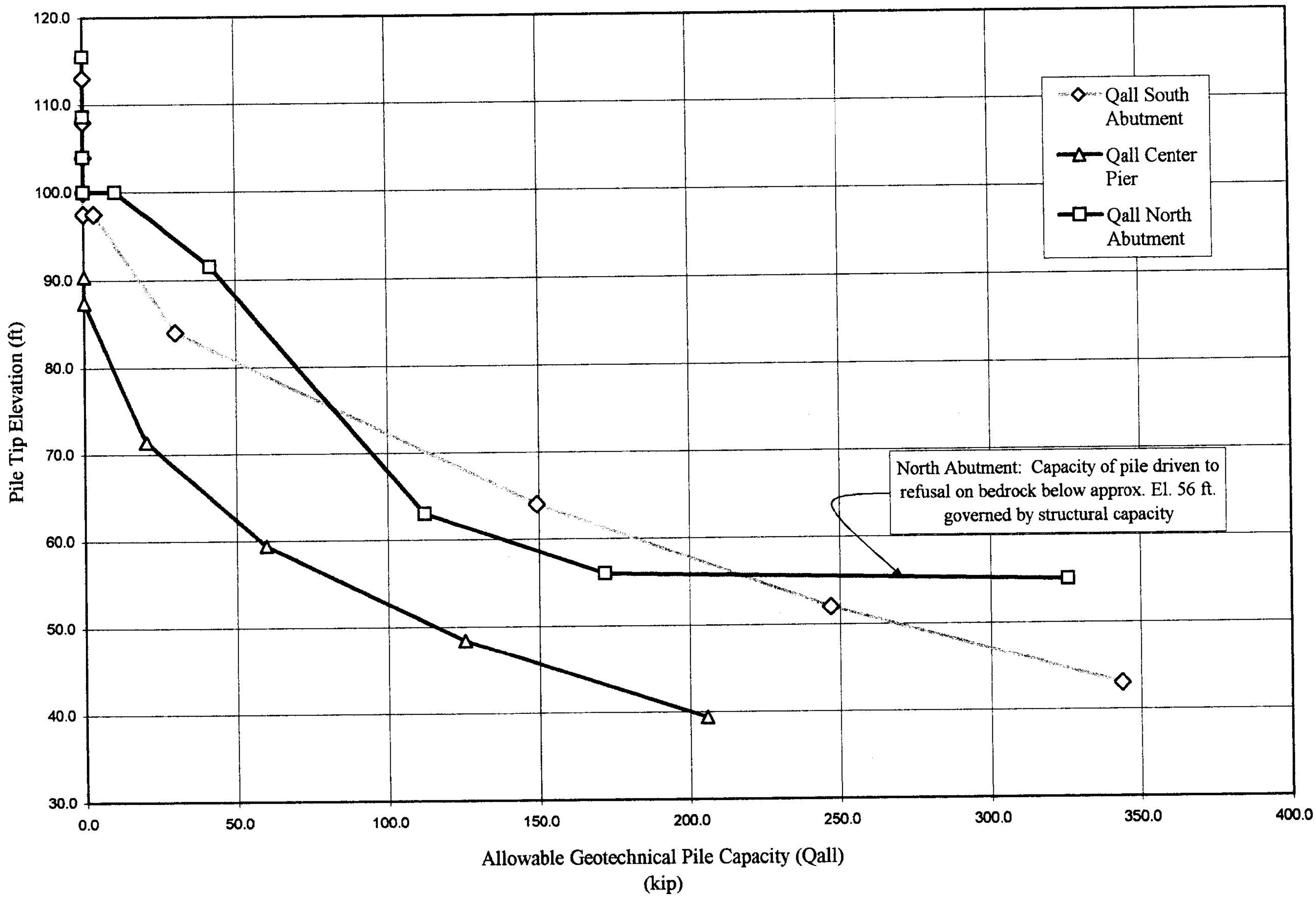
Depth	gamma	PHI	Cd	Pd Avg		delta	Kd	Cf	dZ	Qsi	Qs	Qs
ft	pcf	deg.	ft	psf					ft	lb	lb	kip
0.01	125	0	4.76	1.25		0.00	0	0	0.01	0	0	0
3	125	0	4.76	188.42		0.00	0	0	2.99	0	0	0
3.01	125	36	4.76	689.54		29.23	1.44	0.91	0.01	20.9817	20.9817	0.02098
19	125	36	4.76	689.54		29.23	1.44	0.91	15.99	33549.7	33570.7	33.5707
19.01	130	38	4.76	1596		30.86	1.73	0.9	0.01	60.6009	33631.3	33.6313
31	130	38	4.76	1596		30.86	1.73	0.9	11.99	72660.5	106292	106.292
31.01	135	40	4.76	2400.9		32.48	2.01	0.88	0.01	108.437	106400	106.4
42	135	40	4.76	2400.9		32.48	2.01	0.88	10.99	119173	225573	225.573
42.01	135	42	4.76	3126.9		34.10	2.01	0.87	0.01	145.783	225719	225.719
51	135	42	4.76	3126.9		34.10	2.01	0.87	8.99	131059	356778	356.778

Summary of Results

Geotechnical Pile Capacity
Irving Bridge, Center Pier

Elev 0=90.4	Depth (ft)	Rt (kip)	Rs	Qu	Qall Center Pier
90.4	0.01	0.0	0.0	0.0	0.0
87.4	3	0.0	0.0	0.0	0.0
87.4	3.01	0.0	0.0	0.0	0.0
71.4	19	11.2	33.6	44.7	19.9
71.4	19.01	16.9	33.6	50.5	22.4
59.4	31	28.3	106.3	134.6	59.8
59.4	31.01	40.3	106.4	146.7	65.2
48.4	42	56.3	225.6	281.9	125.3
48.4	42.01	89.9	225.7	315.6	140.3
39.4	51	106.0	356.8	462.8	205.7

Figure A: Allowable Geotechnical Capacity for HP 14x89 Pile at Irving Bridge by Nordlund Method



Attachment D
From Reference 2

would allow the use of a box area at H pile toe and total pipe cross section area for open end pipe pile.

STEP 8 Compute ultimate pile capacity, Q_u (kN).

$$Q_u = R_s + R_t$$

STEP 9 Compute allowable design load, Q_a (kN).

$$Q_a = \frac{Q_u}{\text{Factor of Safety}}$$

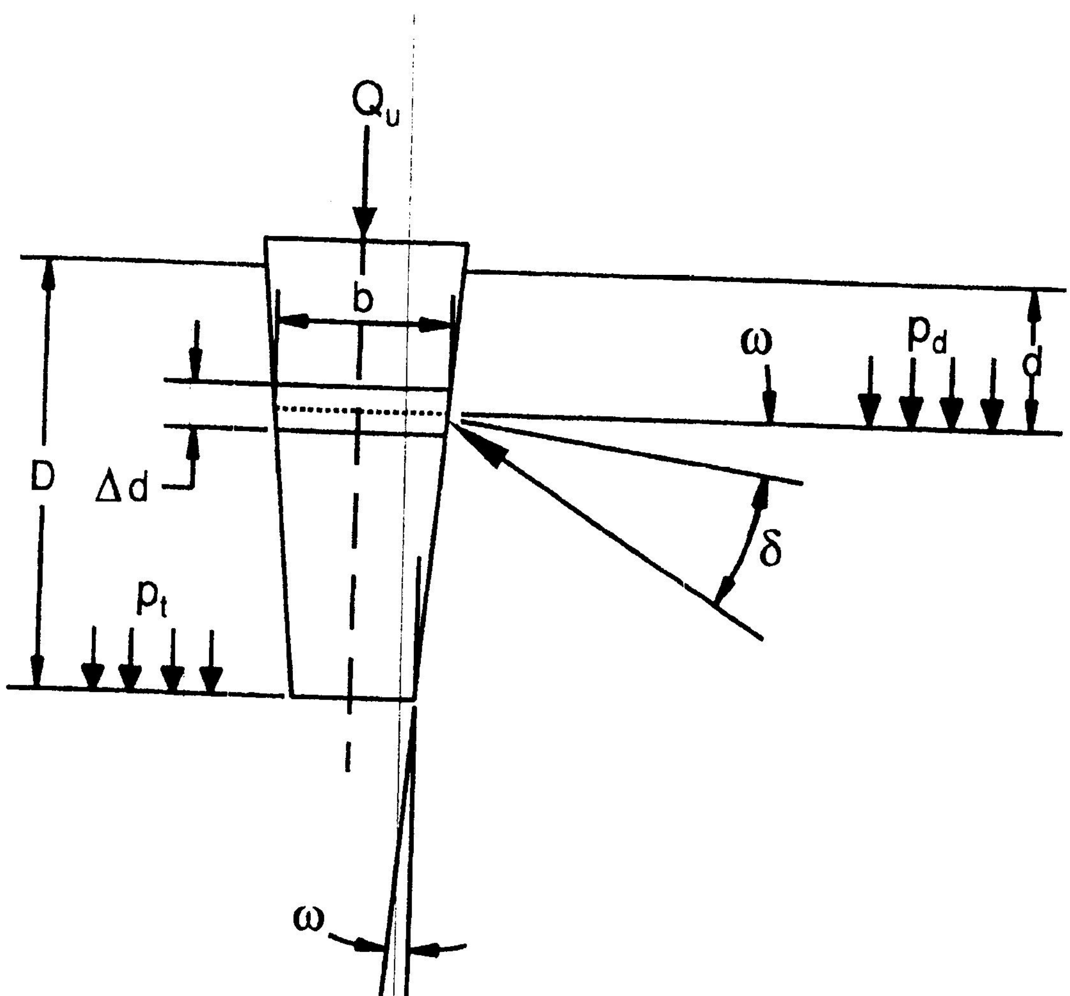
Use Factor of Safety based on the construction control method specified as described in Section 9.6.

In using the Meyerhof method, it should be remembered that it is intended to be used only for preliminary capacity and length estimates. Limiting values often apply for the unit shaft and toe resistances and they should be used. It should also be remembered that the Standard Penetration Test is subject to many errors. Thus, judgment must be exercised when performing capacity calculations based on SPT results.

9.7.1.1b *Nordlund Method*

The Nordlund Method (1963) is based on field observations and considers the shape of pile taper and its soil displacement in calculating the shaft resistance. The method also accounts for the differences in soil-pile coefficient of friction for different pile materials. The method is based on the results of several load test programs in cohesionless soils. Several pile types were used in these test programs including timber, H, closed end pipe, Monotubes and Raymond step taper piles. These piles, which were used to develop the method's design curves, had pile widths generally in the range of 250 to 500 mm. The Nordlund Method tends to overpredict pile capacity for piles with widths larger than 600 mm.

According to the Nordlund Method, the ultimate capacity, Q_u, of a pile in cohesionless soil is the sum of the shaft resistance, R_s and the toe resistance, R_t. Nordlund suggests the shaft resistance is a function of the following variables:



$$Q_u = \sum_{d=0}^{d=D} K_\delta \, C_F \, p_d \, \frac{\sin(\delta+\omega)}{\cos\omega} \, C_d \, \Delta d + \alpha_t \, N'_q \, A_t \, p_t$$

Figure 9.9 Nordlund's General Equation for Ultimate Pile Capacity

STEP 3 Determine the coefficient of lateral earth pressure, K_δ, for each ϕ angle.

a. Determine K_δ for ϕ angle based on displaced volume, V, and pile taper angle, ω, using either Figure 9.11, 9.12, 9.13, or 9.14 and the appropriate procedure described in Step 3b, 3c, 3d, or 3e.

b. If the displaced volume is 0.0093, 0.093, or 0.930 m^3/m which correspond to one of the curves provided in Figures 9.11 through 9.14 and the ϕ angle is one of those provided, K_δ can be determined directly from the appropriate figure.

c. If the displaced volume is 0.0093, 0.093, or 0.930 m^3/m which correspond to one of the curves provided in Figures 9.11 through 9.14 but the ϕ angle is different from those provided, use linear interpolation to determine K_δ for the required ϕ angle. Tables 9-2a and 9-2b also provide interpolated K_δ values at selected displaced volumes versus ϕ angle for uniform piles (ω = 0°).

d. If the displaced volume is other than 0.0093, 0.093, or 0.930 m^3/m which correspond to one of the curves provided in Figures 9.11 through 9.14 but the ϕ angle corresponds to one of those provided, use log linear interpolation to determine K_δ for the required displaced volume. An example of this procedure may be found in Appendix F.2.1.2. Tables 9-2a and 9-2b also provide interpolated K_δ values at selected displaced volumes versus ϕ angle for uniform piles (ω = 0°).

e. If the displaced volume is other than 0.0093, 0.093, or 0.930 m^3/m which correspond to one of the curves provided in Figures 9.11 through 9.14 and the ϕ angle does not correspond to one of those provided, first use linear interpolation to determine K_δ for the required ϕ angle at the displaced volume curves provided for 0.0093, 0.093, or 0.930 m^3/m. Then use log linear interpolation to determine K_δ for the required displaced volume. An example of this procedure may be found in Appendix F.2.1.2. Tables 9-2a and 9-2b also provide interpolated K_δ values at selected displaced volumes versus ϕ angle for uniform piles (ω = 0°).

STEP 9 Compute the ultimate toe resistance, R_t (kN).

a. $R_t = \alpha_t N'_q A_t p_t$

b. limiting $R_t = q_L A_t$

q_L value is obtained from:

1. Entering Figure 9.17 with ϕ angle near pile toe determined from laboratory or in-situ test data.

2. Entering Figure 9.17 with ϕ angle near the pile toe estimated from Table 4-5 and the average corrected SPT N' near toe as described in Step 7.

c. Use lesser of the two R_t values obtained in steps a and b.

For steel H and unfilled open end pipe piles, use only steel cross section area at pile toe unless there is reasonable assurance and previous experience that a soil plug will form at the pile toe. Additional discussion on plug formation in open pile sections is presented in Section 9.10.5. The assumption of a soil plug would allow the use of a box area at H pile toe and total pipe cross section area for open end pipe pile.

STEP 10 Compute the ultimate pile capacity, Q_u (kN).

$$Q_u = R_s + R_t$$

STEP 11 Compute the allowable design load, Q_a (kN).

$$Q_a = \frac{Q_u}{\text{Factor of Safety}}$$

The factor of safety used in the calculation should be based upon the construction control method to be specified. Recommended factors of safety were described in Section 9.6.

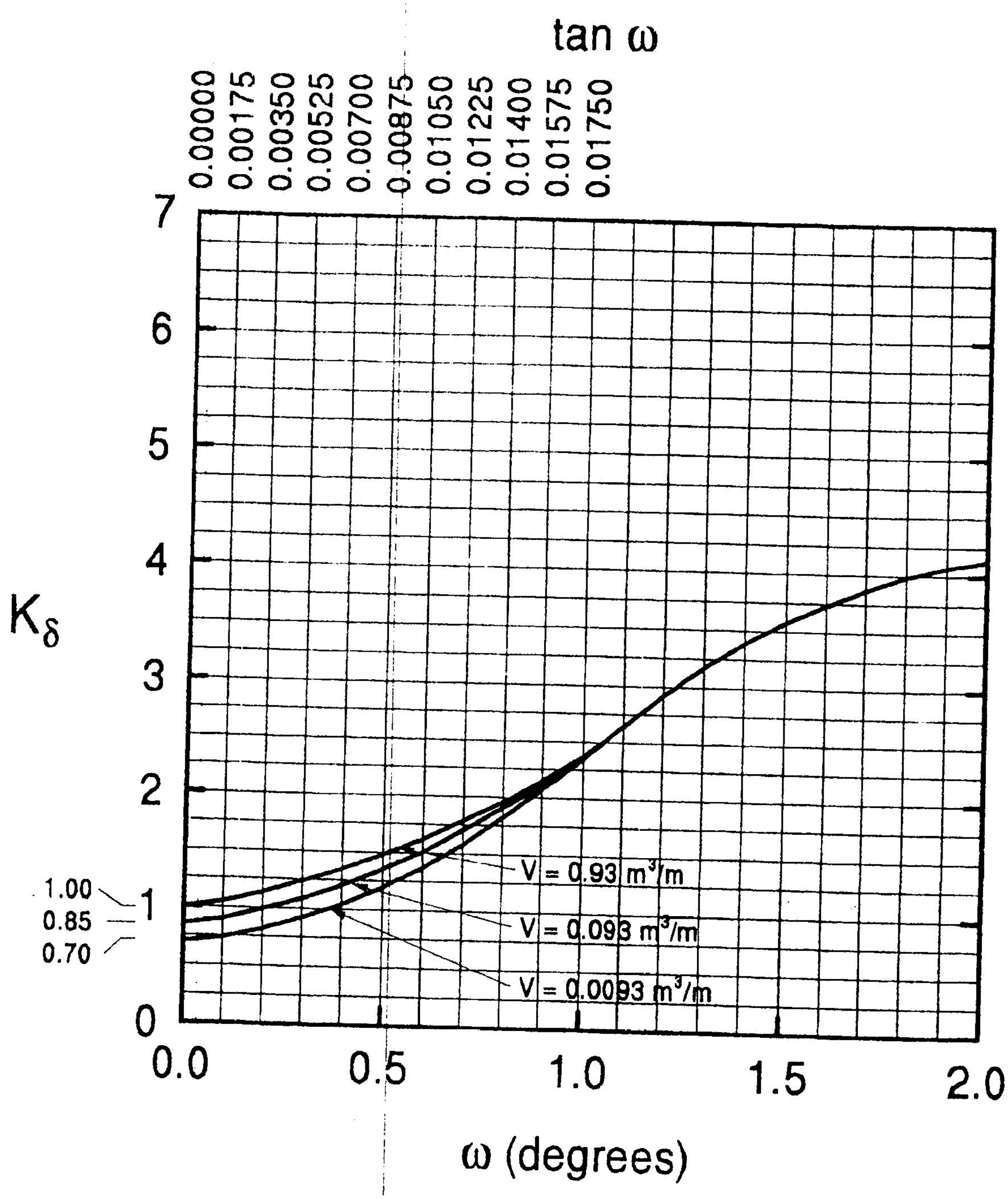


Figure 9.11 Design Curve for Evaluating K_δ for Piles when ϕ = 25° (after Nordlund, 1979)

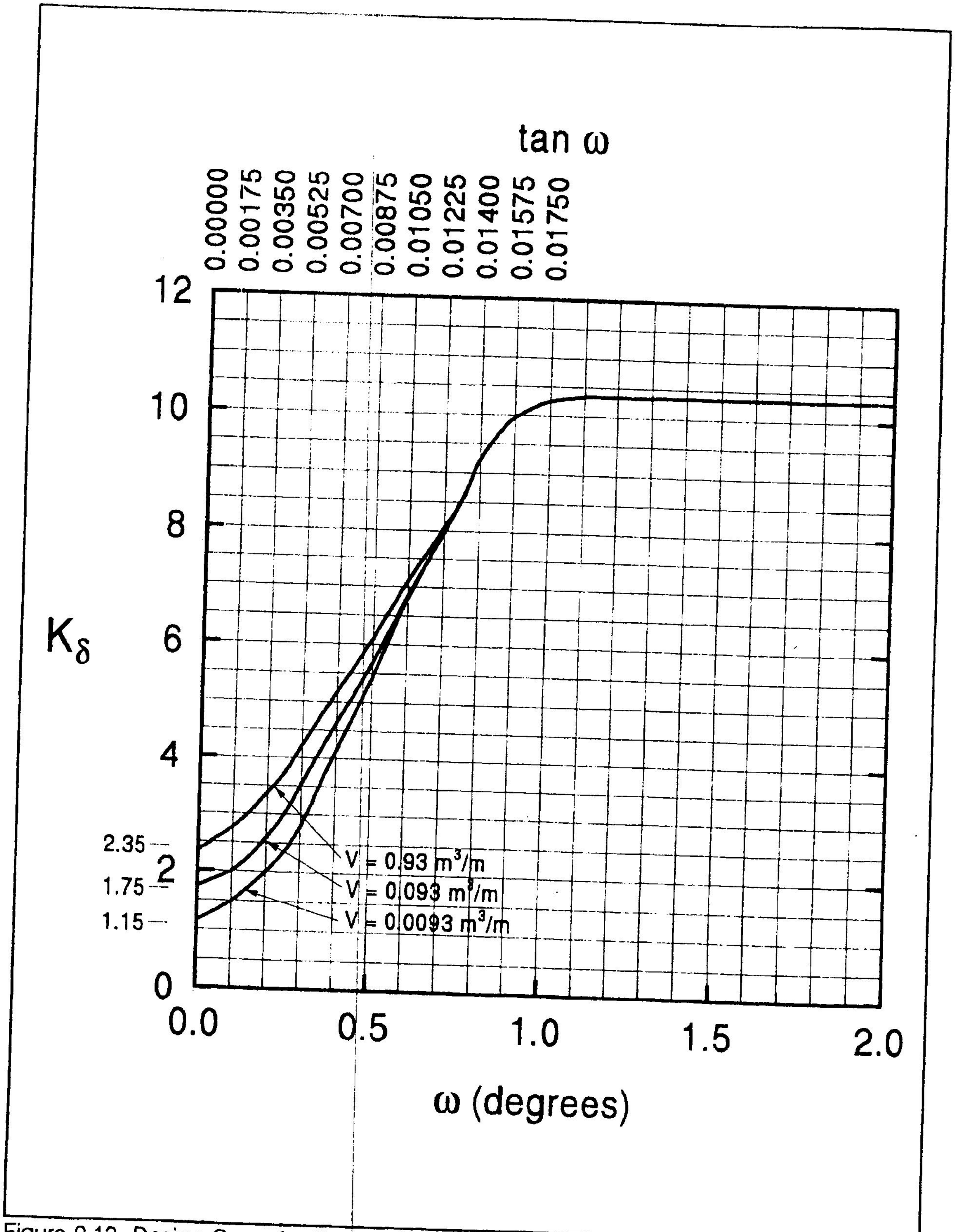


Figure 9.13 Design Curve for Evaluating K_δ for Piles when ϕ = 35° (after Nordlund, 1979)

Table 9-2(a) Design Table for Evaluating K_δ for Piles when $\omega = 0°$ and $V = 0.0093$ to $0.0930\ m^3/m$

ϕ	Displaced Volume (V), m^3/m									
	0.0093	0.0186	0.0279	0.0372	0.0465	0.0558	0.0651	0.0744	0.0837	0.0930
25	0.70	0.75	0.77	0.79	0.80	0.82	0.83	0.84	0.84	0.85
26	0.73	0.78	0.82	0.84	0.86	0.87	0.88	0.89	0.90	0.91
27	0.76	0.82	0.86	0.89	0.91	0.92	0.94	0.95	0.96	0.97
28	0.79	0.86	0.90	0.93	0.96	0.98	0.99	1.01	1.02	1.03
29	0.82	0.90	0.95	0.98	1.01	1.03	1.05	1.06	1.08	1.09
30	0.85	0.94	0.99	1.03	1.06	1.08	1.10	1.12	1.14	1.15
31	0.91	1.02	1.08	1.13	1.16	1.19	1.21	1.24	1.25	1.27
32	0.97	1.10	1.17	1.22	1.26	1.30	1.32	1.35	1.37	1.39
33	1.03	1.17	1.26	1.32	1.37	1.40	1.44	1.46	1.49	1.51
34	1.09	1.25	1.35	1.42	1.47	1.51	1.55	1.58	1.61	1.63
35	1.15	1.33	1.44	1.51	1.57	1.62	1.66	1.69	1.72	1.75
36	1.26	1.48	1.61	1.71	1.78	1.84	1.89	1.93	1.97	2.00
37	1.37	1.63	1.79	1.90	1.99	2.05	2.11	2.16	2.21	2.25
38	1.48	1.79	1.97	2.09	2.19	2.27	2.34	2.40	2.45	2.50
39	1.59	1.94	2.14	2.29	2.40	2.49	2.57	2.64	2.70	2.75
40	1.70	2.09	2.32	2.48	2.61	2.71	2.80	2.87	2.94	3.00

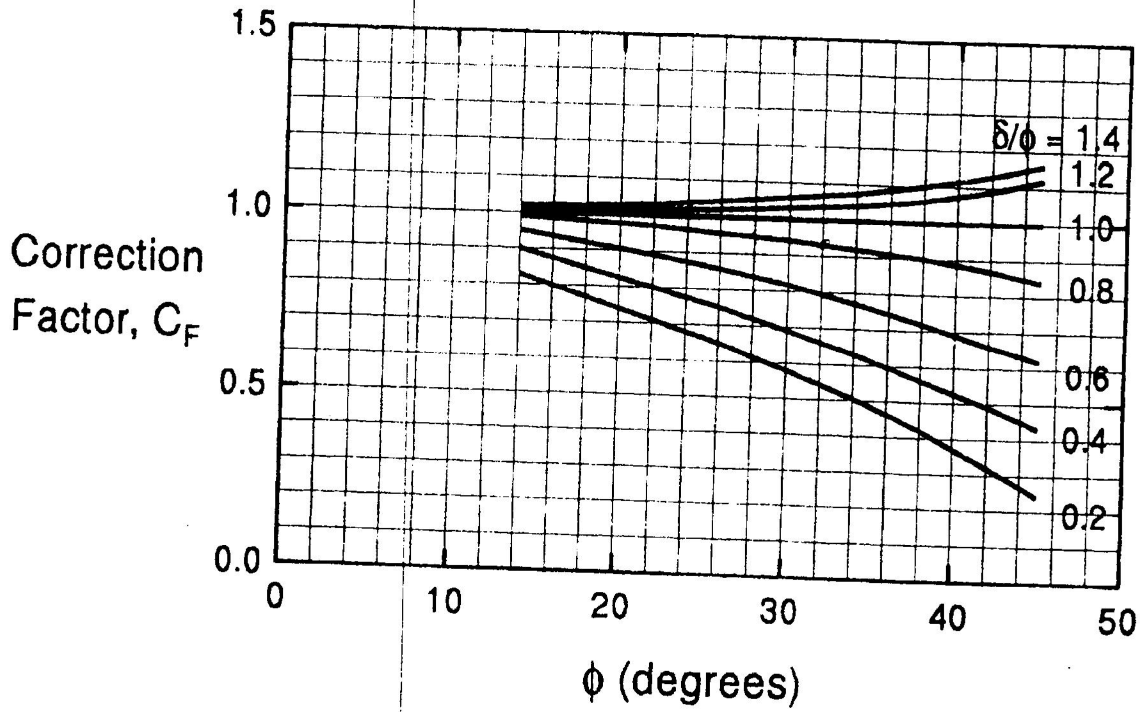


Figure 9.15 Correction Factor for K_δ when $\delta \neq \phi$ (after Nordlund, 1979)

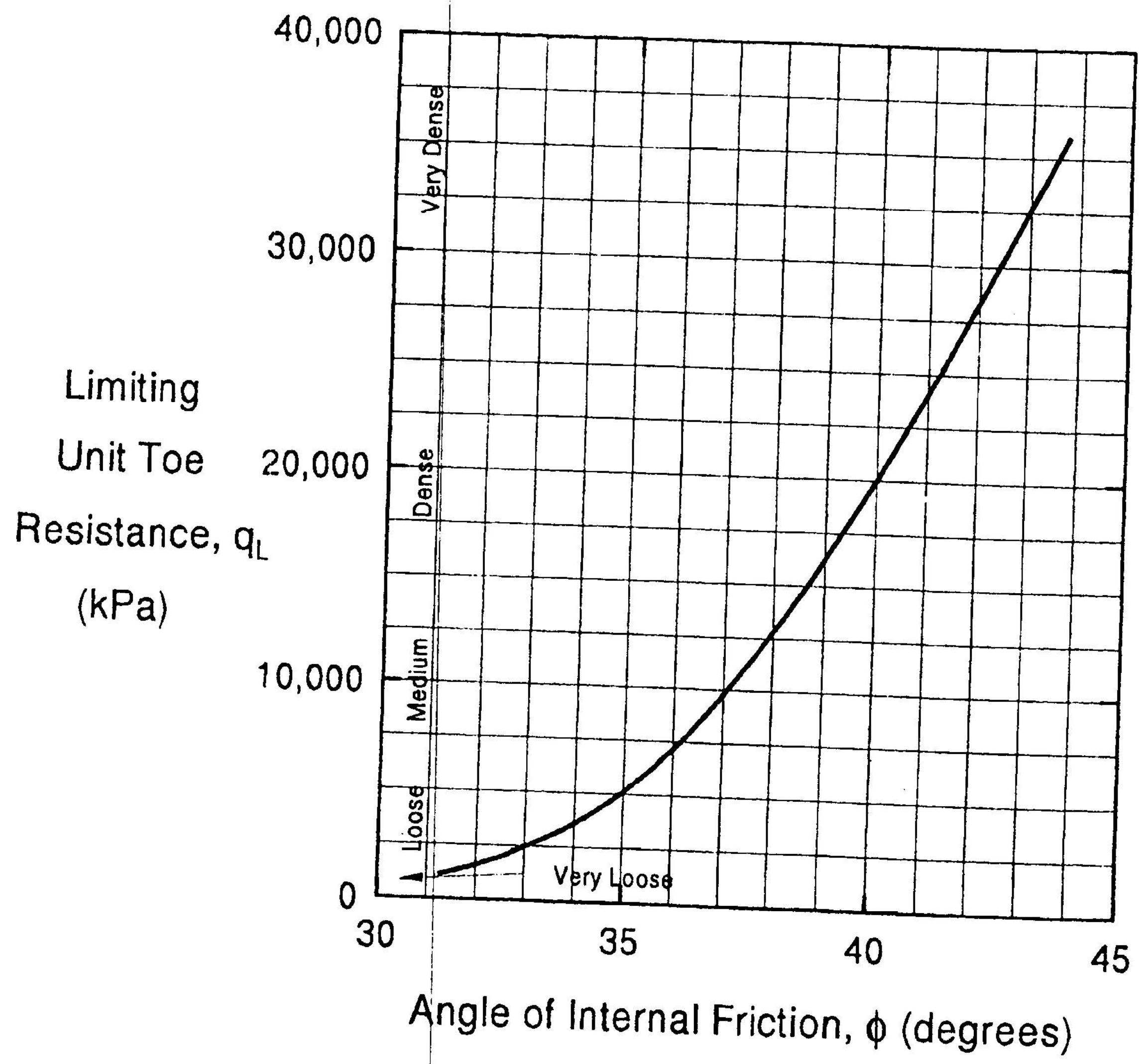


Figure 9.17 Relationship Between Maximum Unit Pile Toe Resistance and Friction Angle for Cohesionless Soils (after Meyerhof, 1976)

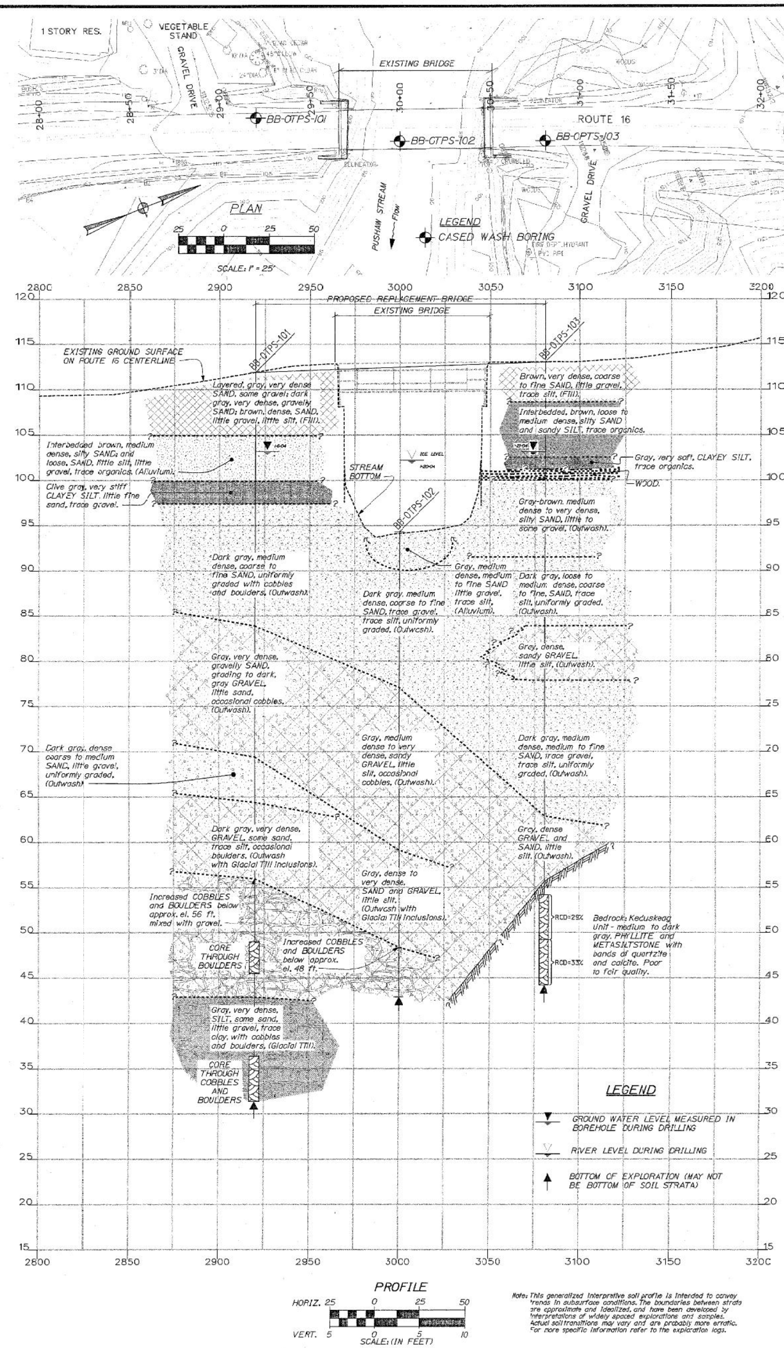
APPENDIX D

FULL SIZE DRAWINGS

SHEET 1

FOUNDATION SURVEY

Filename: ...\GEOTECH\MSTA\001_BLP&SP.dgn Division: GEOTECH Username: kate.maguire Date:8/16/2005



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

PROJECT No.		043-6811
DESIGN	MSP	05-28-04
CADD	RDS	05-28-04
CHECK	MSP	8-24-05
REVIEW	DAE	8/24/05

Golder Associates
Yarmouth, Maine

SHEET NUMBER
1
OF -

IRVING BRIDGE
PUSHAW STREAM
OLD TOWN PENOBSCOT COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

PROJ. MANAGER JOEL KITTREDGE | BY | DATE
DESIGN-DETAILED
CHECKED-REVIEWED K.Maguire 7/13/05
DESIGN2-DETAILED2
DESIGN3-DETAILED3
REVISIONS 1 T.White, MDOT 7/13/05
REVISIONS 2
REVISIONS 3
REVISIONS 4
FIELD CHANGES

SIGNATURE
P.E. NUMBER
DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1104(300)X

BRIDGE NO. 2405
PIN 11043.00
BRIDGE PLANS

SHEET 2

BORING DETAILS

Filename: BORING LOG SHEETS.dgn Division: - Username: - Date:9/1/2005

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project:	Irving Bridge Replacement crossing Pushaw Stream	Location:	Route 16, Old Town, Maine	Boring No.:	BB-OTPS-101
				PIN:	11043.00
Driller:	Northeast Diamond Drilling	Elevation (ft.)	111.3 ft	Auger ID/OD:	
Operator:	R. Leonard	Datum:	NGVD	Sampler:	SPT split spoon, 2 ft. long
Logged By:	R. Bennett	Rig Type:		Hammer Wt./Fall:	140 lb / 30 in
Date Start/Finish:	1/5/04 - 1/8/04	Drilling Method:	Washed, Driven Casing	Core Barrel:	NQ, 2.0 in ID, wire line

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
	1D	18/10	0.5 - 2.0	34/49/65	83	Cased	111.3	6 in. Asphalt pavement	
	2D	24/12	2.0 - 4.0	42/18/52/75	70		109.8	Gray, very moist, very dense, coarse to fine SAND, some gravel, trace silt, with petroleum odor. (Road Subbase Fill) / Dark gray, very moist, very dense, gravelly coarse to medium SAND, trace silt, with petroleum odor. (Road Subbase Fill)	
5	3D	24/12	4.0 - 6.0	35/23/21/41	44		107.8	Olive brown, very moist, dense, medium to fine SAND, little gravel, little silt. (Suspected Subgrade Fill)	
	4D	24/10	6.0 - 8.0	24/12/8/7	20		104.8	Olive brown, wet, medium dense, silty fine to medium SAND, trace fine gravel with occasional mottling. (Alluvium)	
	5D	24/6	8.0 - 10.0	7/3/25/6	28		103.8	Brown, wet, loose, coarse to fine SAND, little to some silt...	

Boring No.: BB-OTPS-101

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project:	Irving Bridge Replacement crossing Pushaw Stream	Location:	Route 16, Old Town, Maine	Boring No.:	BB-OTPS-102
				PIN:	11043.00
Driller:	Northeast Diamond Drilling	Elevation (ft.)	94.4	Auger ID/OD:	
Operator:	C. Palmer	Datum:	NGVD	Sampler:	SPT split spoon, 2 ft. long
Logged By:	R. Bennett	Rig Type:	CME model 75	Hammer Wt./Fall:	140 lb. / 30 in.
Date Start/Finish:	1/20/04-1/21/04	Drilling Method:	Washed, Driven Casing	Core Barrel:	NQ, 2.0 in ID, wire line

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
	1D	24/4	0.0 - 2.0	5-6-8-7	16	17		Gray, wet, medium dense, medium to fine SAND, little gravel to 3/4 in., trace silt. (Alluvium)	
5	2D	24/11	5.0 - 7.0	10/8/7/6	15	40	90.4	Dark gray, wet, medium dense, coarse to fine SAND, trace fine rounded gravel...	

Boring No.: BB-OTPS-102

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project:	Irving Bridge Replacement crossing Pushaw Stream	Location:	Route 16, Old Town, Maine	Boring No.:	BB-OTPS-103
				PIN:	11043.00
Driller:	Maine DOT	Elevation (ft.)	112.9	Auger ID/OD:	
Operator:	C. Mann	Datum:	NGVD	Sampler:	SPT split spoon, 2.0 ft. long
Logged By:	R. Bennett	Rig Type:	CME Model 45c	Hammer Wt./Fall:	140 lb. / 30 in.
Date Start/Finish:	0830 1/20/4 - 1500 1/21/4	Drilling Method:	Drive and Wash	Core Barrel:	NQ, 2.0 in. ID, wire line
Boring Location:	Proposed north abutment	Casing ID/OD:	4.0 in.	Water Level*:	9.62 ft. bgs 1/21/04

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
							112.6	5 in. asphalt pavement.	
	1D	24/6	2.5 - 4.5	65/50/33/15	83			Brown, moist, very dense, coarse to fine SAND, little fine gravel, trace silt. (Road subbase fill)	
5	2D	24/24	5.0 - 7.0				108.5	Olive brown, very moist, loose, silty fine SAND, trace...	

Boring No.: BB-OTPS-103

PROJECT No.	043-6811		
DESIGN	MSF	05-2804	
CADD	RDE	05-2804	
CHECK	WSP	8-24-05	
REVIEW		7/24/05	

Golder Associates
Yarmouth, Maine

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BH-1104(300)X

BRIDGE NO. 2405 PIN NO. 11043.00 BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	-	-
DESIGN-DETAILED	-	05/27/04
CHECKED-REVIEWED	-	-
DESIGN2-DETAILED2	-	-
DESIGN3-DETAILED3	-	-
REVISIONS 1	-	-
REVISIONS 2	-	-
REVISIONS 3	-	-
REVISIONS 4	-	-
FIELD CHANGES	-	-

SIGNATURE

P.E. NUMBER

DATE

IRVING BRIDGE
PUSHAW STREAM
OLD TOWN PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER

2

2 OF 2